Laboratory Performance Test for Asphalt Concrete

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Eshan V. Dave, Principal Investigator
Department of Civil Engineering
University of Minnesota Duluth

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16. Abstract (Limit: 250 words)

The asphalt mixture design and acceptance procedures for Minnesota Department of Transportation are currently governed primarily by the mixture composition requirements put forth through use of various volumetric measures (such as, air content, asphalt film thickness, aggregate gradation etc.). The asphalt binder has been required to meet performance criteria through the Superpave asphalt binder specifications. This study looked at use of laboratory performance test for asphalt mixtures. The study was conducted in three phases, first phase focused on merging the asphalt mix design records with the pavement performance data to determine effects of mix design parameters on asphalt pavement cracking performance. Second and third phase used a series of field sections across Minnesota to conduct field performance evaluations as well as laboratory tests on field cored samples. The testing for second and third phase of the study focused on using disk-shaped compact tension (DCT) fracture energy test as a laboratory performance test. The findings form he first phase of study indicated that the asphalt binder type as defined by the Superpave performance grade (PG) plays an important role in affecting the field cracking performance, majority of mixture design parameters did not indicate a consistent effect on field cracking performance, this reinforces the need for use of laboratory performance test as a mixture design tool as well as acceptance parameter. The DCT testing results showed trends consistent with previous and other on-going research studies, whereby the asphalt mixtures with higher fracture energies corresponded with pavements with lower amount of transverse cracking.

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Prepared by:

Eshan V. Dave
Chelsea E. Hanson
Benjamin Helmer
Jay Dailey
Chelsea M. Hoplin
Department of Civil Engineering
University of Minnesota Duluth

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EXECUTIVE SUMMARY

This report provides the summary of research efforts and corresponding findings from all four tasks of the research study on laboratory performance test for asphalt concrete (MnDOT contract no. 99008 work order 40). Task-1 of this study undertook the effort of developing a comprehensive database that includes the asphalt mix design records as well as the field cracking performance for all asphalt pavements in Minnesota that are under the jurisdiction of MnDOT. The database was thereafter used to extract a number of datasets to conduct statistical analyses. The primary objective of these analyses were twofold: (1) determine whether the indirect tensile strength (ITS), measured using the modified Lottman procedure (AASHTO T-283), can be used as a pavement cracking performance measure; and, (2) determine the effects of mix design parameters (asphalt binder type and amount, asphalt film thickness, voids in mineral aggregates and presence of recycled materials) on pavement cracking performance. The results from this study conclude that the ITS parameter is not recommended for use as a performance measure. A number of mix design parameters showed a significant effect on the field cracking performance. Both asphalt binder grade and binder amounts had significant effect on field cracking performance. Use of softer asphalt binders (-34 versus -28 low temperature grades) lead to a greater fraction of pavements without cracking. Higher amounts of binder content supported lower amount of pavement cracking. The asphalt film thickness (AFT) and voids in mineral aggregates (VMA) both showed a statistically significant effect on pavement cracking performance. Greater percentages of crack free pavements were found for mixes with lower AFT and mixes with higher VMA. The presence of recycled materials in asphalt mixes may cause greater fraction of pavements to undergo cracking; however, the amount of data that was available for pavements with “all virgin” materials was limited and thus this conclusion is preliminary in nature. Task-2A of the study focused on field cracking performance evaluation. The field visits also aided in development of sampling plans for cored sample procurement to be used for Task-2B. Task-2B consisted of disk-shaped compact tension fracture energy testing on the field samples of each section. Task-3A used the field performance evaluations from Task-2A to make comparisons of the mix design parameters to various transverse cracking performance measures that were developed through this study. Similarly Task-3B used the disk-shaped compact tension (DCT) fracture energy to make comparisons to the mix design parameters. Fracture energy showed the most correlation to the asphalt binder properties of the mix, i.e. PG grade, PG spread, and PG low-temperature grade. The last task of this study (Task-4) assessed the effects performance based specification implementation as well as drafted these specifications. The final outcome of this project is in the form of added confidence in the use of asphalt mixture fracture energy using the DCT test as a transverse cracking performance parameter. At present, parallel efforts are underway with other research studies for a full-scale implementation of these specifications (MnDOT Contract 99008, Work Order 162). The testing requirements for these specifications follow the ASTM D7313-13 with modifications to enhance test practicality and repeatability.

CHAPTER 1: INTRODUCTION

1.1 Background and Motivation

At present, the MnDOT material specifications for plant produced asphalt mixtures (MnDOT 2360) do not require performance-based laboratory tests as part of the mix acceptance criteria. This leads to a high risk for inferior pavement performance. A preceding MnDOT synthesis study on this topic further reinforced the need for asphalt performance testing. The study also found that several State highway agencies have adopted performance testing requirements. This formed the basis of the current study that focuses on identification of a laboratory asphalt performance test and evaluates the effects of various asphalt mix design parameters on pavement cracking performance.

1.2 Organization of the Report

- The research study was organized into four tasks with subtasks:
- Task-1: Analysis of Laboratory Test and Field Performance Data (Task-1) (Chapter 2);
- Task-2A: Cracking Performance Evaluation of Field Sections (Task-2A);
- Task-2B: Laboratory Testing (Task-2B);
- Task-3A: Analysis of field performance data (task-3a) and Laboratory Testing Results (Task-3B)(Chapter 5);
- Task-3B: Analysis of field performance data (task-3a) and Laboratory Testing Results (Task-3B) (Chapter 5); and
- Task-4: Implementation Assessment and Draft Performance Based Specifications (task-4) (Chapter 6).

The Task-1 undertook the effort of developing a comprehensive database that includes the asphalt mix design records as well as the field cracking performance for all asphalt pavements in Minnesota that are under the jurisdiction of MnDOT. The database was thereafter used to extract a number of data sets to conduct statistical analyses. The primary objective of these analyses were twin fold: (1) Determine whether the indirect tensile strength (ITS), measured using the modified Lottman procedure (AASHTO T-283), can be used as a pavement cracking performance measure; and, (2) Determine the effects of mix design parameters (asphalt binder type and amount, asphalt film thickness, voids in mineral aggregates and presence of recycled materials) on pavement cracking performance.

The Task-2A involved field evaluation of several roadways across Minnesota. Nine asphalt roadways were chosen for this study through interactions with the technical advisory panel (TAP) for the project. The projects were chosen to obtain a wide cross-section of varying asphalt mixture designs and pavement structures. During the course of this task construction plans were evaluated, site visits were conducted, field cracking performance was determined and field sampling plans were developed. This portion will provide an overview of the individual site visits, cracking performance information for each pavement section and the field sampling plans.

The Task-2B of the "project involved laboratory testing of field cored samples from several roadways across Minnesota. During the course of this task field samples were tested using the disk-shaped compact tension test (DCT). This task provided a comparison of cracking performance and laboratory performance test results.

The main purpose of the Task-3A and 3B is to determine if any correlation exists between mix design properties with field performance and laboratory performance testing. Thus, this task serves as a check or validation for the general findings made through Task-1. Visual distress surveys were conducted by researchers and performance was quantified using various developed cracking measures. Laboratory testing results were compared with various mix design properties and preliminary correlations between performance testing and mix parameters evaluated.

Lastly, the main purpose of the Task-4 was to propose draft performance-based specifications that include a requirement for a disk-shaped compact tension (DCT) fracture energy laboratory test as part of mix design and acceptance process. The scope also included evaluation of the fracture energy thresholds (limits) proposed by earlier studies. While the current MnDOT DCT implementation research study (Work Order 162) is undertaking full scale implementation tasks the findings and efforts from the present study initiated that effort and aided in formalization of the aforementioned study's work plan. The topics evaluated in the present work includes sampling requirements, specimen preparation requirements, equipment costs and manpower requirements.

It is strongly recommended that Task-2A and 3A sections as well as Task-2B and 3B sections be read in conjunction with each other. Task-2A and 2B provide background on preliminary observations and findings related to the performance analysis in Task-3A testing conducted in Task-3B.

CHAPTER 2: ANALYSIS OF LABORATORY TEST AND FIELD PERFORMANCE DATA (TASK-1)

2.1 Introduction

2.1.1 Overview of Task-1

The task-1 of the project consists of review of previous and current MnDOT projects to extract material testing and pavement performance data. The majority of material property data was obtained from MnDOT's LIMS (laboratory information management system) and the pavement performance data was obtained from MnDOT's Office of Pavement Management. The extracted data was used to conduct a series of statistical analysis to determine correlations between pavement performance and properties of asphalt mixtures.

The research efforts began with the tensile strength ratio (TSR) measurements of over 2000 mixtures that are available from MnDOT's Office of Materials and Road Research. The data consists of dry and conditioned (wet) indirect tensile strength (ITS) measurements. Statistical analyses were conducted to determine significance of ITS measurements with the asphalt mix design parameters (such as, binder grade, volumetric properties etc.) and field performance. This analysis helped determine the suitability of ITS as performance indicator. The mix parameters were also correlated to field cracking performance. This analysis helped determine the effect of various mix design choices on the pavement cracking performance. This report provides the summary of research conducted for the task-1 of the project.

2.1.2 Organization of Task-1

The task-1 is organized in five sections. The description of various databases that were used for this study along with the methodology for construction of a single comprehensive database that consists of material properties and pavement performance data are provided in section 2.2. The overview of statistical analysis procedures and the schemes used to present the results are also discussed in 2.2. The results for analysis of ITS and TSR data are presented in section 2.3. Section 2.4 provides analysis of asphalt mix design parameters in context of field cracking performance. Finally the summary, conclusions and recommendations are presented in section 2.5.

Along with this report a series of electronic files are also delivered. The electronic files include: Comprehensive database in Microsoft Access and analysis files and result outputs from statistical software SAS. The comprehensive database has all the material property and pavement performance data and. Due to large amount of data that has been used in this task and large amounts of statistical analysis conducted, it was not possible to provide all the results in this report. However, all the analysis results as well as raw data are available in the electronic files accompanying this report.

2.2 Data Sources, Research Methodology, and Result Presentation Scheme

2.2.1 Introduction

This section provides the overview on the various data sources that were used in this study, the methodology that was followed for construction of a comprehensive database and for analysis of various data sets generated using the aforementioned database, and finally description of various schemes used in this report to present the results of the data analysis. The methodology described in this chapter can be followed by future researchers to conduct additional data analysis using the electronic data set that is accompanying this report.

2.2.2 Data Sources

The data sources that were combined to construct a comprehensive database of asphalt mix design parameters and pavement field cracking performance were obtained from MnDOT's Office of Materials and Road Research (OM&RR). The data was received in the form of Microsoft Excel spreadsheets. The data sources consist of four primary data sources. These data sources are comprised of: (1) Mixture Design Reports (MDR); (2) Laboratory Information Management Systems (LIMS); (3) Tensile Strength Ratio (TSR); and, (4) Pavement Management Systems (PMS). It should be noted that the TSR data was previously extracted from the LIMS data source.

As can be seen in Table 2.1: Data Sources and Amount of Available Data obtained from MnDOT, the data sources are listed along with the number of records that each data set contained. The amount of available data represents a record kept by the MnDOT during the asphalt mix testing or distress survey on the pavement. Different records are located in each data source. For example, cracking amounts are available in the PMS data set while the mix design parameters (AFT, VMA, etc.) are located in the LIMS data set. Some data sets contain larger amounts of data based on record keeping conventions. For example, multiple samples were taken from a specific project in the LIMS data set. The PMS data set contains such a large amount of records due to distresses being recorded on the same pavement section over multiple years.

Table 2.1: Data Sources and Amount of Available Data obtained from MnDOT

Data Source	Amount of Available Data
Material Design Recommendation	12,293
LIMS Database	32,515
TSR Database	2,545
Pavement Management Systems	58,416

Construction of a comprehensive database consisting of all information from all sources was necessary to evaluate the effects of various variables on field performance and on each other as well. Recording the mix design data and having no way to analyze how the mixes are performing in the field gives no feedback as to how to improve the overall mix design process. This database gives MnDOT the ability to extract records from multiple sources and conduct a statistical analysis on the effect of mix parameters on either strength of the mixes or field cracking performance. It can also be utilized in future efforts to analyze and track asphalt mix designs and field performance.

2.2.2.1 MDR Data

The MDR data consists of asphalt mix designs that were submitted for approval before they were accepted to be used for placement in the field. As can be seen in Table 2.1 the MDR dataset contains 12,293 records of data. The range of years this data was recorded is 2001 to 2012. Information on mixes containing recycled materials is also found in this data set. A search in the MDR data source for mixes containing recycled materials, such as recycled asphalt pavement (RAP), recycled asphalt shingles (RAS), and Millings, returned 1,039 records. Using both RAP and RAS in asphalt concrete mixes is a relevant means to incorporate materials that would otherwise be waste into an asphalt pavement mix design. Using these waste materials can add a sustainable aspect to the mix design process. While reuse of material might lower the cost of the mix and can add a sustainable aspect, it is important to evaluate the effects of recycled materials on pavement performance to determine if the resulting mixes are truly sustainable in nature or not. A statistical analysis of how these mixes perform in the field, in terms of amount of cracking, was investigated and will be discussed later.

2.2.2.2 LIMS Data

The LIMS data source consists of mix design information recorded during the pavement construction as part of QA/QC procedure. The LIMS data source is the only source that contains mix design information. This data source is a crucial part of the database due to this reason. Without this information there would be no way to analyze the effect of mix design parameters on mix strength, TSR, or field performance. The data ranges over 2004 to 2012.

The mix design information includes what will be referred to as mix parameters. It is common to use acronyms for these parameters. An explanation of these acronyms and the definitions of various mix parameters used in this study is as follows:

- PG Grade – Superpave Performance Grade of asphalt binder. The high temperature represents an average seven day maximum temperature and the low temperature represents the lowest expected temperature of the pavement surface temperature.
- PG LT- the low temperature of the PG Grade
- PG Spread- The sum of the high temperature and low temperature ratings of the PG Grade. This represents the range of temperature difference for which the binder is graded.
- AFT – Asphalt Film Thickness is an estimate of the thickness of binder coating aggregate.
    - AFT P_{be} – A function of effective binder and surface area of aggregate. The surface area is determined using the gradation and estimate surface area factors for aggregates in each sieve size range.
    - AFT Adjusted- A function of effective binder and surface area of aggregate in sample as well as specific gravity of aggregates. The surface area of aggregates is based on the gradation. The calculated surface area is adjusted according to the specific gravity of aggregates.

- Air Void Level- Amount of air voids present in an asphalt sample.
    - Design Air Void Level – Percentage of air voids that is selected for design of asphalt mix.
    - Actual Air Void Level – Actual air voids measured from a lab tested sample (typically collected during the mix production or from existing pavement).
- VMA- Voids in Mineral Aggregate represents the volume fraction of air voids and effective asphalt binder in the mix.
    - VMA Ignition –Asphalt binder percentage present in a sample obtained by use of an ignition oven. The asphalt binder percentage is then used to calculate the VMA.
    - VMA Extraction- Asphalt binder percentage present in a sample obtained by extracting with use of chemicals. The asphalt binder percentage is then used to calculate the VMA.
- VFA – Voids Filled with Asphalt represents the percent of VMA that is occupied by asphalt binder.
- Design Traffic Level- Level assigned to mixes based on the traffic level as expressed by 20 year equivalent single axle loads (ESALs).
- Percent Binder- Percentage of asphalt binder present in asphalt mix.
    - Percent Binder Ignition- Asphalt binder percentage present in a sample obtained by use of an ignition oven.
    - Percent Binder Extraction- Asphalt binder percentage present in a sample obtained by extracting with use of chemicals.

2.2.2.3 TSR Data

The TSR data source consists of records of the indirect tensile strength (ITS) of different mixes from the AASHTO T 283 (Modified Lottman Test) test. The ITS values are available for mixes with and without moisture conditioning, typically referred to as “dry” and “wet” ITS. From these strengths, the TSR of the mix is then found. This test is typically conducted during the mix design acceptance process as a screening test to ensure that asphalt mix is not susceptible to moisture induced damage. For the mix to be acceptable it must have a minimum TSR of 75% for traffic levels 2 and 3 and 80% for traffic levels 4 and 5. Mix design parameters combined with the wet strength, dry strength, and TSR were statistically analyzed to determine if the AASHTO T 283 test can be used as a laboratory performance test for prediction of field cracking performance. These results will be discussed further in this report. The data contained in the TSR data source range from year 2000 to 2011.

2.2.2.4 PMS Data

The PMS data source is comprised of both pavement section information as well as distress data. The pavement section information is defined in terms of beginning and ending reference posts as well as beginning and ending total mileage, or GIS coordinates. A Log Point Listing form is used by the MnDOT district office to convert GIS points to reference posts. These conversions were done prior to obtaining the records.

The PMS data source contains all of the field performance (distress) data, specifically cracking performance of different pavement sections. Information pertaining to route types (Interstates, State highways, and US highways) and route numbers are included in this data source which contains 188 unique routes. The distress information includes transverse cracking, longitudinal cracking, rutting, raveling, patching, and longitudinal joint deterioration. Due to the main focus of this study pertaining to cracking of asphalt pavements, only transverse and longitudinal cracking were included in the statistical analysis phase. Inclusion of this data source into the database allows for the ability to track the effect of different mix design parameters on field performance of the pavement over several years. This data contains information recorded between year 2004 and 2011.

The transverse and longitudinal cracking data in the PMS data is collected based on the severity of the cracks, namely low, medium and high. For each severity level the data is reported in terms of percent cracking (% cracking) which is calculated as 2 times the number of cracks per 500 feet length of the survey section. For purposes of conducting a statistical analysis between amount of cracking and laboratory tests as well as asphalt mix parameters, a number of measures of field cracking performances can be calculated. In this study, the researchers looked at transverse and longitudinal cracking amounts in two primary ways: (1) total cracking; and, (2) total weighted cracking. Total cracking is sum total of low, medium and high severity cracks, whereas weighted cracking amount is arbitrary cracking amount with weight factors of 1, 2 and 4 applied to low, medium and high severity crack amounts.

The total cracking and total weighted cracking amounts for a given PMS section for each year of distress survey can be used to calculate additional cracking measures that are representative of field cracking performance. These measures for transverse cracking are described in Table 2.2.

Table 2.2: Field Cracking Measures

Measure	Description	Unit
Maximum Total Transverse Cracking Amount (MTCTotal)	Maximum transverse cracking amount (low + medium + high) of all survey years for a pavement section normalized against number of years for which pavement section has been in service.	% cracking/year
Maximum Total Weighted Transverse Cracking Amount (MTCWeighted)	Maximum weighted transverse cracking amount (low + 2*medium + 4*high)/6 of all survey years for a pavement section normalized against number of years for which pavement section has been in service.	% cracking/year
Maximum Total Transverse Cracking Rate (MTCRTotal)	Maximum increase in total transverse cracking amounts (low + medium + high) between any two consecutive years of service.	% cracking/year
Maximum Total Weighted Transverse Cracking Rate (MTCRWeighted)	Maximum increase in total weighted transverse cracking amounts (low + 2*medium + 4*high)/6 between any two consecutive years of service.	% cracking/year
Average Total Transverse Cracking Rate (ATCTotal)	Difference between maximum and minimum total transverse cracking amounts (low + medium + high) divided by number of years that pavement section has been in service.	% cracking/year
Average Weighted Total Transverse Cracking Rate (ATCWeighted)	Difference between maximum and minimum total weighted transverse cracking amounts (low + 2*medium + 4*high)/6 divided by number of years that pavement section has been in service.	% cracking/year

Measures similar to those shown in Table 2.2 can be used for longitudinal cracking. These are namely, MLCTotal, MLCWeighted, MLCRTotal, MLCRWeighted, ALCTotal and ALCWeighted.

2.2.2.5 GIS Data

It should also be noted that Geographic Information Systems (GIS) records obtained from MnDOT were also used in constructing the comprehensive database. This data source was needed due to it containing the State Project (SP) number for each pavement section, which is a key piece of information that was instrumental in linking the field performance data back to the mix parameter data sources. A total of 1,321 records from 1999-2012 were included in this data source.

2.2.3 Data Mapping

The computer software program used in this study to compile and build the comprehensive database of both mix design parameters and field performance data was Microsoft Access. This software program allows for importing different sets of data, such as Microsoft Excel Spreadsheets, and combining or "linking" the multiple data sets together into one comprehensive database. The five data sources described previously are imported as "Tables" into Access. The combination of these tables into one Access file creates a comprehensive database. This newly formed database allows for vital information from different sources to be combined together in one list. This list of data can then be used for analysis. For this study, asphalt mix design parameters from the LIMS data source were combined with ITS, TSR, and field performance data for conducting statistical analysis.

The "linking" of data sources will be referred to as "data mapping" throughout the rest of the report. The records that are common across various data sets are used for "linking" them. These common records are referred to as "mapping parameters". These are the means by which multiple data sources are combined together to allow for specific information to be extracted from each source. They also allow for traversing throughout multiple data sets when looking for specific mix parameters or field performance quantities. Searches conducted within Access for certain parameters are referred to as queries.

2.2.3.1 Data Mapping Parameters

For the multiple data sets that are imported into Access, it is crucial to have as many defining mapping parameters in common as possible. Having multiple ways to link the data sets together refines the results when a query is conducted in Access. When mapping parameters are linked within Access, only information that both tables have in common will be returned upon running the query. The mapping parameters used from each data source are listed in Table 2.3. Without these mapping parameters there would be no way to link all of the data sources together, and no database could be built.

Table 2.3: Mapping parameters related to different data sources

Data Source	Mapping Parameter
Material Data Records	MDR, Mix Design
LIMS Database	Project Number (SP), MDR , Mix Design
TSR Database	Project Number (SP), Mix Design
Pavement Management Systems	Route Type, Route Number, Year, Pavement Section Reference Points
Geographic Information Systems	Route Type, Route Number, Year, Pavement Section Reference Points, Project Number (SP)

The PMS as well as the GIS data sources have multiple mapping parameters in common. These include route type, route number, and pavement section reference points. These two data sources were combined into one source that contained both field performance cracking distress information as well as the Project Number, which was from the GIS data source. It was crucial to combine these two sources due to the project number being needed to link the distress information back to the mix parameter data sources. An explanation of how this combination of PMS and GIS data into one source is explained in section 2.3.3.

2.2.3.2 Data Mapping within Microsoft Access

The method of using mapping parameters to join data sources in Microsoft Access is visualized in Figure 2.1 Joining of Mapping Parameters between Data Sources in Microsoft Access. This screen shot illustrates how a query is conducted in Microsoft Access. The two tables selected in this example are the LIMS and TSR data sources. These tables are joined by mapping parameters in order to link the two sources together. This linking will allow for only information that is common between the two tables to be returned upon running the query. Mapping parameters of project number and mix designator (indicated as SP# in the figure) are shown in this example as the links between the two data sources. Connecting the two data sources by these mapping parameters allows for a query to be run that returns a specific information from each source. For example, a search can be done that returns mix parameters (PG Grade, Percent Binder, etc.) from the LIMS data source that are paired with their corresponding ITS based on using project numbers and mix designators as mapping parameters between the two sources.

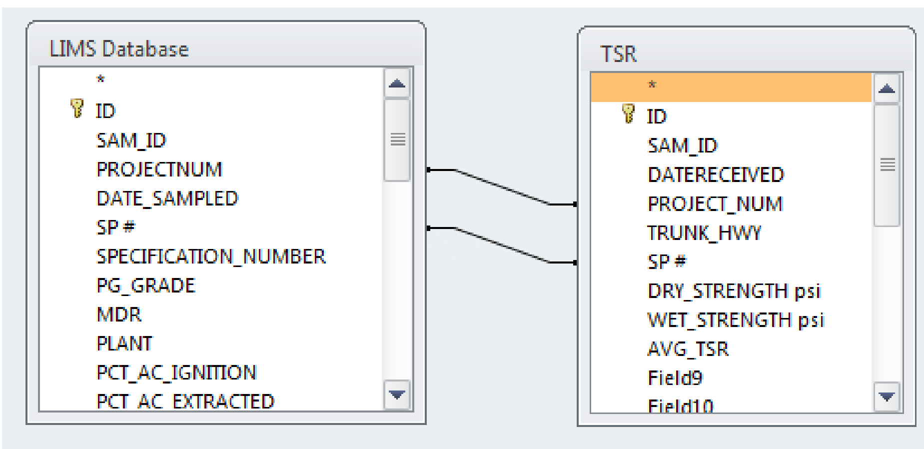

Figure 2.1 Joining of Mapping Parameters between Data Sources in Microsoft Access

The linking of LIMS and TSR data sources is shown in Figure 2.2. This schematic again shows that the two tables are linked together by use of Project Number and Mix Designator as mapping parameters. The final outcome from this linking and combining is also shown in Figure 2.2. From this combined data set, lists of specific mix design information and corresponding ITS or TSR of that mix can be returned.

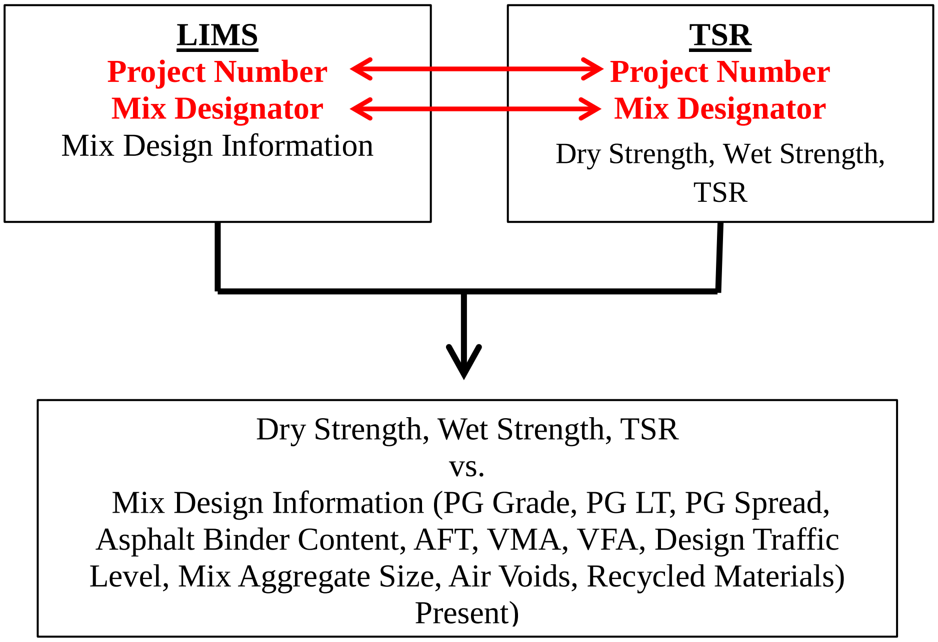

Figure 2.2: Linking and combining of LIMS and TSR data sets in Microsoft Access

2.2.3.3 Data Mapping using Custom Algorithm

For cracking distress data that was combined and analyzed with mix design information, the PMS data source needed to contain a mapping parameter that is common with both the LIMS and TSR data sources. The PMS data source contains distress information, as can be seen in Table 2.3, and is the only source of field cracking data in the comprehensive database. Without a common mapping parameter between the TSR, LIMS, and PMS data sources, no analysis of cracking data with mix parameters could be conducted.

The GIS data source does however have a mapping parameter in common with the LIMS and TSR data sources. The Project Number (SP) of various mixes was included in GIS information. Both TSR and LIMS also contain the Project Number (SP) as a mapping parameter, as seen in Table 2.3. There was a need to combine the PMS and GIS data in order for the Project Number of various mix designs to be joined with cracking distress information. Use of Microsoft Access for such combination was not possible as the PMS evaluation sections are often evenly spread across highway and do not directly overlap with scope of a pavement construction project, which is the case with GIS and in-turn LIMS and TSR databases.

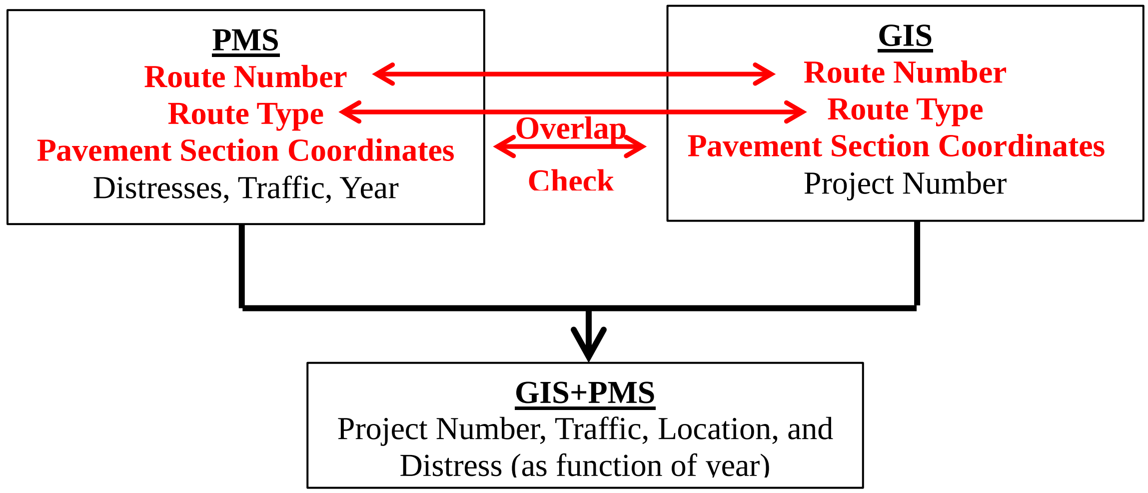

Figure 2.3: Linking and combining of PMS and GIS data sets using Custom Algorithm

Both PMS and GIS data sources contained common mapping parameters such as route number, route type, and pavement section coordinates. These pavement section coordinates however did not match up perfectly from one data source to another. This is due to the scope of pavement construction and rehabilitation projects to be independent from the PMS distress survey sections. An algorithm was created in Visual Basic within Microsoft Excel in order to combine the GIS and PMS data. The combined data was generated using the GIS coordinates of pavement sections.

The algorithm convention was to traverse through each record in the GIS dataset and for that record identify all the overlapping pavement sections in the PMS database. Once the exact match in GIS and PMS records had checked for the same route number, route type, and pavement section coordinate within the PMS source, the program then went to the next GIS record and continually looped through the program until every record was checked. Four different scenarios were possible while checking if the pavement section coordinates overlapped between the PMS and GIS data sources. These scenarios are shown schematically in Figure 2.4. The highlighted areas denote areas of overlap between pavement sections from the GIS and PMS data sources. Scenarios 3 and 4 also contained an additional check to ensure the overlap length of the pavement was at least 10% of total length of the GIS or PMS pavement section. If after the program was run and it was determined the pavement sections did match, the GIS records along with cracking information was combined on a table. This allowed for the cracking distress data to contain the Project Number mapping parameter.

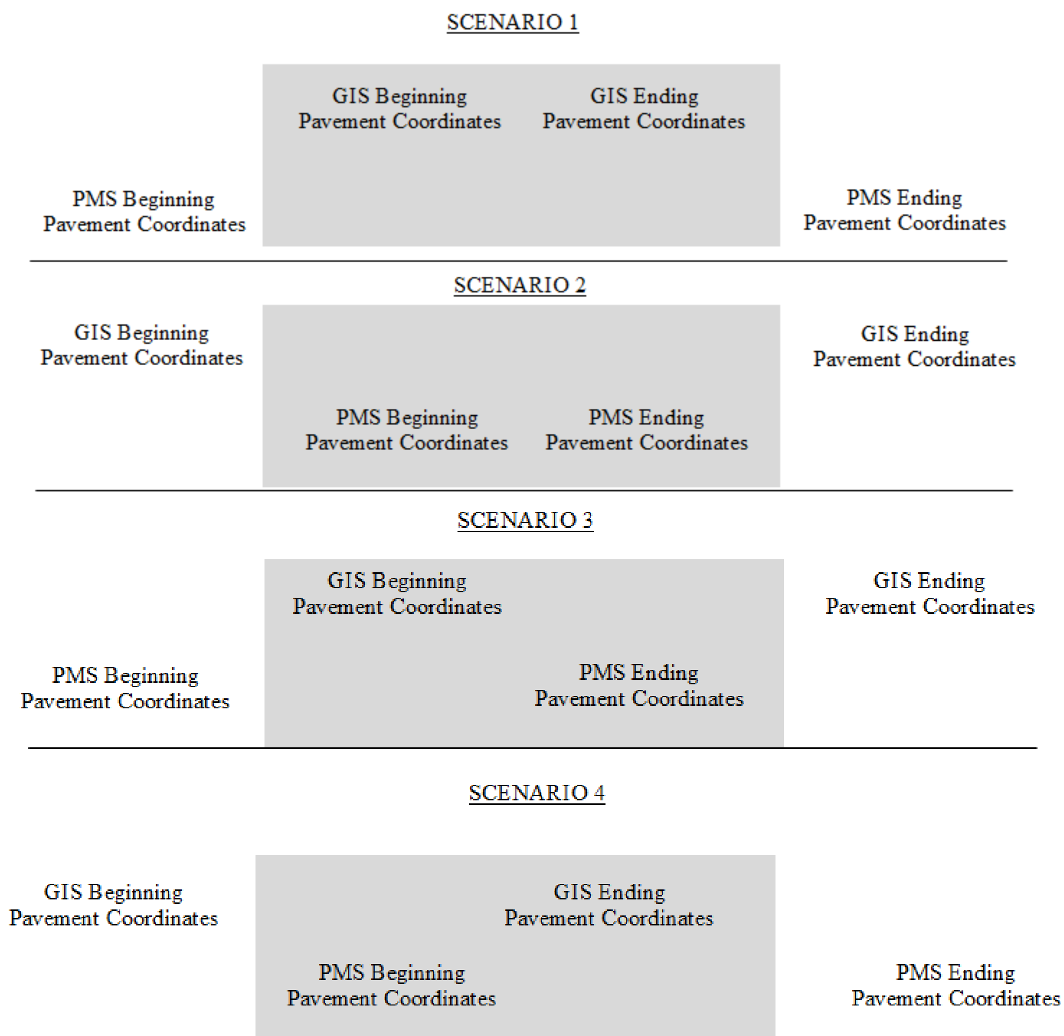

Figure 2.4: Pavement section coordinate overlap scenarios

Before the combination of GIS and cracking records could be done, a check of both distress survey years and years of newest rehabilitation or maintenance efforts on the corresponding pavement sections needed to be completed. The PMS data source contained information on years during which distress surveys were conducted. A large number of records exist in this data source due to distress surveys being conducted and recorded for multiple years for the same pavement section. The GIS data source contains the project let date, or most recent date of maintenance or rehabilitation construction.

Within the Visual Basic program, logic was implemented to screen out only records from the distress surveys that were conducted during the year of roadway construction or after the year of roadway construction. Distress data recorded before the latest construction year was screened out due to the cracking data not accurately corresponding to the asphalt material properties available in database. The range of years of construction let dates within the GIS data source is 1999-2012.

The ranges of years for which distress surveys are available in PMS data source range from 2004-2011. This results in records from 1999-2003 from the GIS data source to be screened out of the cracking analysis. Overall, the cracking analysis was conducted on pavements from the years 2004-2012.

Both transverse and longitudinal cracking data is recorded in the combined GIS and PMS data source. Within the Visual Basic code, six different cracking measures were calculated and recorded. These are same as those described in Table 2.2, that is, Maximum Total Transverse Cracking Amount (MTCTotal), Maximum Total Weighted Transverse Cracking Amount (MTCWeighted), Maximum Total Transverse Cracking Rate (MTCRTotal), Maximum Total Weighted Transverse Cracking Rate (MTCRWeighted), Average Total Transverse Cracking Rate (ATCTotal), and Average Weighted Total Transverse Cracking Rate (ATCWeighted) for transverse cracking. Similar measures are recorded for the longitudinal cracking. All of these amounts were recorded and included on a new data sheet with both GIS and field cracking data. The total amount of GIS and cracking records returned after running the macro was 2,128.

Once this combined sheet of cracking data was recorded, it was then imported back into Microsoft Access to be included in the database. This allowed for the cracking data to be linked with the LIMS, TSR, and MDR data sources by the Project Number mapping parameter. This is schematically shown in

Table 2.5. From this combined data source, queries were conducted to combine cracking amounts and cracking rates with various mix parameters. These lists of data were then statistically analyzed, which are discussed later in this report.

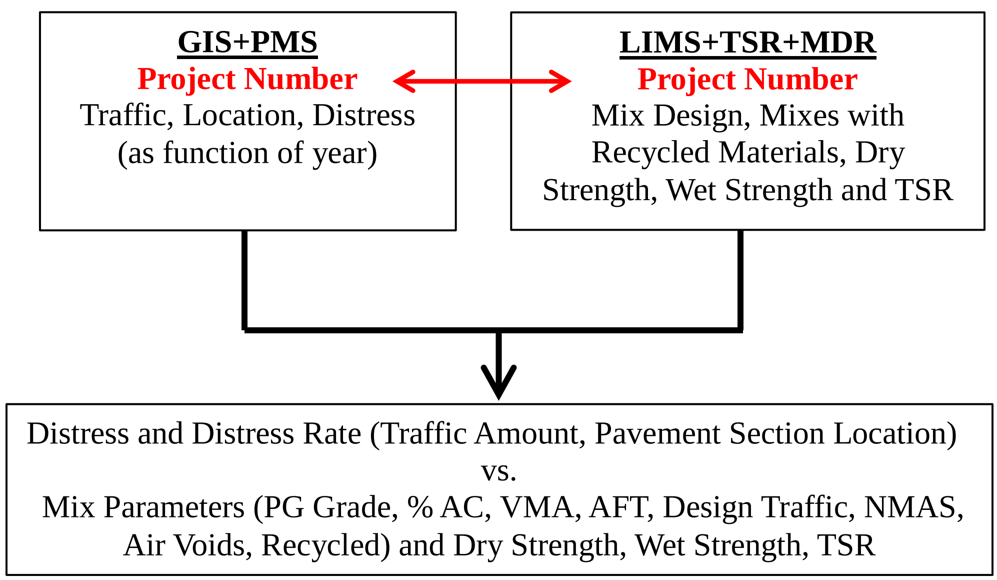

Figure 2.5: Linking and combining of PMS and GIS datasets to LIMS, TSR and MDR data.

2.2.4 Data Analysis Methodology

As described in the previous section, a comprehensive database of both mix design parameters and field performance cracking information was built in order to analyze if a statistically significant relationship existed between these measures. A statistically significant relationship would illustrate that certain mix design parameters have an effect on field cracking performance. Whether mix parameters and field cracking performance were related to ITS (dry or wet), or TSR of various mixes was also investigated. Significant relationships between mix design parameters and field cracking performance and the ITS or TSR is of interest when analyzing the effectiveness of using the AASHTO T-283 test as a field performance measure. Similarly, the effect of mix design parameters on cracking performance can provide information that can help modify mix design requirements and policy decisions, such as recommended asphalt binder grades or allowance for use of recycled materials.

Figure 2.6 provides the schematic of data analysis. Data was exported from the comprehensive database and imported into statistical software titled SAS. A least-square mean and regression analysis was conducted on the data sets. The statistical analysis procedure used in this research is described in this section as analyzing mix design parameters with ITS data, as an example. Similar analyses were conducted for other measures, such as field cracking performance and mix design parameters.

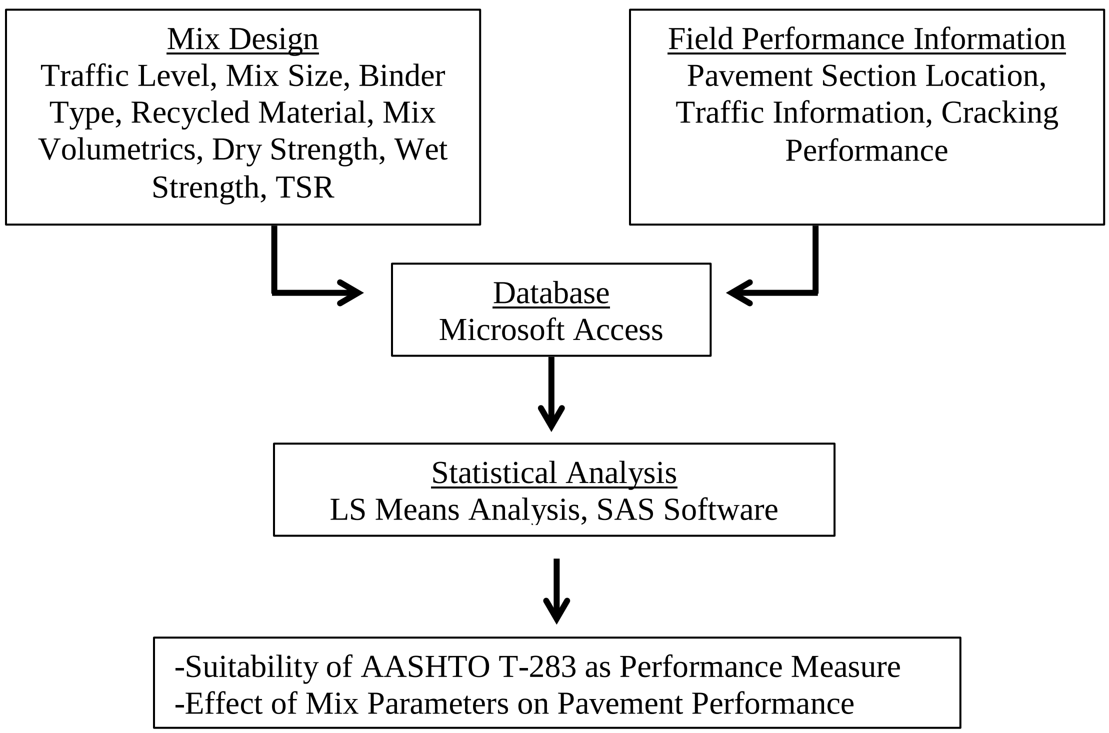

Figure 2.6: Schematic of database and data analysis organization

The statistical analysis dealing with mix parameters, ITS (dry and wet) and TSR was done in two phases. The first phase of statistical analysis includes single variable correlation. This consisted of only one mix parameter being paired with the ITS (dry and wet) and TSR, and then analyzed to investigate if a statistically significant relationship existed between them. The second phase of the analysis was a multiple variable correlation. Based on findings in phase one, groupings of two mix parameters were extracted from database records and returned with their ITS and TSR values. Mix parameters used in both the single and multiple variable analyses will be discussed in Chapter 3 in more depth.

Once the data was exported from the database, it was input to the SAS software. The data was analyzed using the least square means (LS Means) procedure. This type of analysis allows for investigating effects of multiple variables on a parameter of interest. For example, combined effects of asphalt mix design traffic level and asphalt binder grade on the dry ITS of mix.

A screenshot of an output from the SAS software is shown in Figure 2.7. This table contains a variety of statistical and regression outputs based on input data. The value that is of most interest in this study was the p-value from the TYPE III SS results. It is labeled in the lower right hand of the output table in Figure 2.7. The TYPE III SS results were used versus the TYPE I SS due to TYPE III being a partial sum of squares. This means that the variables are being analyzed with all other variables present in the statistical model. With TYPE I, the variables are being added one at a time to the model based on how they were input into the program. This is referred to as a sequential sum of squares. Due to observing how the parameters affect the model as a whole with all other parameters also included, specifically during the multiple variable analysis, the TYPE III output is most relevant.

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	39	2012247.125	51596.080	59.63	<.0001
Error	7919	6852193.158	865.285		
Corrected Total	7958	8864440.283			

R-Square	Coeff Var	Root MSE	DryStrength Mean
0.227002	29.23860	29.41573	100.6058

Source	DF	Type I SS	Mean Square	F Value	Pr > F
PGGrade	8	1842434.403	230304.300	266.16	<.0001
AsphaltContent	6	71981.532	11996.922	13.86	<.0001
PGGrade*AsphaltConte	25	97831.190	3913.248	4.52	<.0001

Source	DF	Type III SS	Mean Square	F Value	Pr > F
PGGrade	8	125732.9492	15716.6187	18.16	<.0001
AsphaltContent	6	13602.0369	2267.0061	2.62	0.0154
PGGrade*AsphaltConte	25	97831.1899	3913.2476	4.52	<.0001

P < 0.05

reject null hypothesis

Figure 2.7: Output from least square mean analysis in SAS of a multiple variable analysis

In statistics, a null hypothesis is used to determine if there exists a relationship between certain variables. The p-value represents if the null hypothesis is rejected or accepted, meaning it is either true or false. It is common practice to utilize a relatively low p-value (< 0.05) for rejecting the null hypothesis. This can also be stated as “there exists a mathematical relationship between variable 1 and variable 2, such that a linear function of variable 1 can predict variable 2 within a 95% confidence interval spread of variable 2 data”. For the analysis conducted in this research, the null hypothesis was that no significant relationship occurred between mix parameters and ITS, TSR, or cracking measures. Thus, a p-value of < 0.05 represents a significant relationship occurring between the mix parameters being tested and either the ITS, TSR, or field cracking measures.

In the example shown in Figure 2.7, the multiple variable analyses containing mix parameters of PG Grade and Asphalt Content were analyzed against ITS (dry) to determine if a statistically significant relationship existed. The p-values for all mix parameters were < 0.05. Both PG Grade and the combined effect of PG Grade and Asphalt Content have p-values of < 0.0001, while Asphalt Content has a p-value of 0.0154. The smaller p-value represents that the significance between the variables is strong. The null hypothesis was rejected and it can be stated that PG Grade, Asphalt Content, and combined effects of PG Grade and Asphalt Content are related to the ITS (dry) of the asphalt mixes in database. It can also be inferred that the PG Grade and combined effects of PG Graded and Asphalt Content are strongly related to ITS (dry), whereas asphalt content is weakly related.

The initial analysis of ITS and TSR data with mix design parameters was conducted using the least squares mean regression analysis (LS Mean). This analysis was sufficient for most mix parameters due to the refinement of the data sets that was done through the use of bounds on the mix parameter values. These bounds allowed the LS Mean procedure to analyze the mix parameters with ITS or TSR. However, when conducting analysis to determine the statistical significance between ITS or TSR and the field cracking performance, it is not possible to put bounds on either set of variables due to the extent of spread. Thus, linear regression was sought as the alternative way to conduct a statistical analysis. Linear regression is based on inputting variables and analyzing if a linear relationship exists between them. An output from a linear regression analysis from the SAS software can be seen below in Figure 2.8. The null hypothesis in a linear regression procedure is that the parameter estimate of the variable is 0. The parameter estimate column is outlined in Figure 2.8. This means that a p-value < 0.05 represents accepting the null hypothesis, and the parameter estimate is significantly different from zero. Thus, concluding that the variable contributes to the linear model of parameter that is being tested. In the example shown in Figure 2.8, ITS (wet) has a p-value < 0.0001. This represents that the parameter (transverse cracking amount) can be expressed as function of ITS (wet). The linear regression output also provides the coefficient of determination (or R^2) which is measure of the quality of fit for the aforementioned linear model. This parameter is important as it provides the measure of reliability with which the parameter (such as field cracking amount) can be predicted using the variable (such as ITS (wet)).

For brevity only concise tables showing the p-values will be included in the rest of the report, however the electronic data set accompanying this report includes the detailed analysis results.

Analysis of Variance					
Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	1	3703.95146	3703.95146	20.17	<.0001
Error	358	65736	183.61954		
Corrected Total	359	69440			

Root MSE	13.55063	R-Square	0.0533
Dependent Mean	16.68097	Adj R-Sq	0.0507
Coeff Var	81.23405		

Very low R-square
Poor quality of fit

Parameter Estimates					
Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Intercept	1	6.59044	2.35746	2.80	0.0055
WetStrength	1	0.12242	0.02726	4.49	<.0001

p-value < 0.05
Accept null hypothesis

Figure 2.8: SAS linear regression output table

2.2.5 Presentation of Results

The data obtained after the statistical analysis will be presented in both graphical and tabular formats that contain information on the statistical inference gained from the SAS analysis as well as from the linear regression. Graphing of the data is shown to illustrate any concentration of data points that occurred as well as the spread of data points. Graphical interpretation is also important as the statistical analysis can often times indicate that one variable (such as, percent binder content) has significant effect on a quantity of interest (such as ITS), but may not point out as to how much change in that variable leads to a significant change in the quantity of interest. Furthermore, the quality of fit in linear regression also provides insight on reliability of using the variable to predict the quantity of interest as well as the general direction of trend.

Data vital to describing statistical relationships and significance will be included in the main body of the report. All other tables and graphs can be found in the electronic dataset that is accompanying this report. More concise tables have been generated for each set of data analyses. An example of such summary table is shown in Table 2.4. The table is showing the parameters that are being compared to ITS. These could be air void level, percent binder content, PG grade, ITS etc. Also notice that the last row of table shows results from grouped or paired analysis (herein referred to as “multiple variable analysis”), whereby combined effect of two parameters on the ITS was evaluated. The table shows three scenarios, the first scenario is for correlation between Parameter 1 and field cracking amount, where a relatively high p-value (> 0.05) indicates that field cracking amount is independent of Parameter 1. The second scenario shows that there exists a weak correlation between field cracking amount and Parameter 2, as evident by p-value that is smaller than 0.05 but not close to zero. Finally, the third scenario is in the last row which indicates that in a combined manner Parameter 1 and Parameter 2 has statistically significant effect on the ITS. Although visually there was no trend, the statistical significance was conducted to see if statistics would support the same thing. The data shows minimal amount of correlation evident by a very low coefficient of determination.

Table 2.4: Example of statistical analysis data table

Mix Parameter	p-value	ITS is related to mix parameters?
Parameter 1	0.231	no
Parameter 2	0.00235	yes
Parameter 1 and Parameter 2	< 0.0001	yes

An example of graphical presentation of the data is presented in Figure 2.9. This graphical data provides information that is supplemental to the information from Table 2.4. The statistical test informed that Parameter 2 has an effect on amount of cracking however did not provide us with additional information such as, how significantly does amount of field cracking change with change in Parameter 2, how reliably the field cracking can be predicted using Parameter 2, and finally whether the data agrees with general engineering knowledge. The plot shown in Figure 2.9 provides this information. It can be observed that the change in field cracking amount is relatively small over a large change in Parameter 2, which in case of this plot is ITS (dry). For change in ITS (dry) of 40 to 200 psi, the field cracking amount increased from 9 to 35 %/500 ft/year. The plot also indicates that the trend is counter-intuitive to general engineering knowledge that greater tensile strength is preferred. Finally, the quality of fit is very poor with coefficient of determination (R^2) to be 0.0474. Thus, this graphical representation was helpful in determining that Parameter 2 should not be used as a pavement performance indicator since: (a) The reliability of predicting performance is low (because of low R^2); (b) The effect of Parameter 2 on amount of cracking is small (small change in amount of field cracking for large change in Parameter 2); and (c) The data trend is reverse of what is expected based on engineering knowledge.

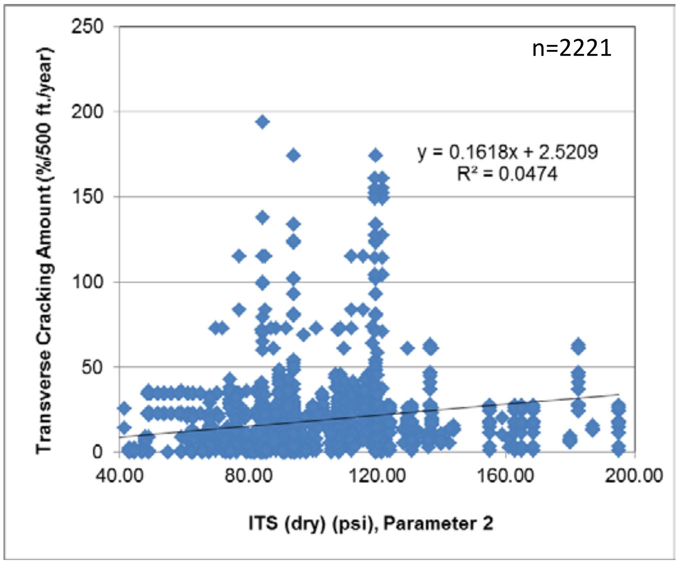

Figure 2.9: Example of graphical data presentation

2.3 Effectiveness of ITS and TSR as Mix Performance Measure

2.3.1 Introduction

This chapter describes the analysis that was conducted to determine suitability of the properties obtained from the AASHTO T-283 test procedure in the form of dry and wet indirect tensile strength (ITS) and/or tensile strength ratio (TSR) as mix performance parameter(s). Since the AASHTO T-283 testing is already part of the current MnDOT 2360 specifications for plant produced asphalt mix, if the material properties from this test can be used as pavement performance measure, minimal additional implementation and testing infrastructure development would be necessary.

As described in Chapter 2, the analysis was conducted in two phases. The first phase evaluated whether various mix design parameters had significant effects on lab measured ITS and TSR. The second phase evaluated the correlation between ITS and TSR with field cracking measures. The evaluation of effects of mix design parameters on ITS and TSR was necessary to determine if ITS and TSR are sensitive to commonly used asphalt mix design controls, such as asphalt film thickness (AFT) or asphalt binder grade.

2.3.2 Effects of Mix Design Parameters on ITS and TSR

This section describes the statistical analysis and the corresponding results for determination of effects of mix design parameters on the ITS and TSR of asphalt mixes. The initial analysis was conducted by evaluating statistical significance of one mix design parameter at a time on ITS and TSR, which is herein referred to as “single variable analysis”. This was followed by a “multiple variable analysis” where combined effects of parameters were evaluated.

2.3.2.1 Single Variable Analysis

The first phase of the statistical analysis dealt with looking into the relationship between single mix parameters (single variable) and ITS (dry), ITS (wet), and TSR of various mixes. The mix parameters extracted by using queries run within the established database are listed in Table 2.5. The definitions of various mix design parameters are provided in section 2.2.2.2 LIMS Data.

Table 2.5: Single variable analysis mix parameters

Mix Parameters	AASHTO T 283 Measurements
AFT (Adjusted and Pbe)	Dry Indirect Tensile Strength (Dry ITS), Wet Indirect Tensile Strength (Wet ITS), Tensile Strength Ratio (TSR).
Actual Air Voids	
NMAS (Aggregate Mix Size)	
Percent Binder (Ignition and Extraction)	
PG Grade	
PG Spread	
PG LT	
Design Traffic Level	
VMA	
VFA	

The data is presented as the scatter plot for purpose of visualizing the breadth of the data and also to show if any visually observable trends were present (or absent). The data was thereafter processed to evaluate normalized frequencies of the ITS and TSR as function of mix design parameter and also to determine the mean, medium and standard deviations. Finally the data was processed through statistical analysis software SAS to determine if there was a statistically significant relationship between the mix parameters and ITS or TSR. The analysis and results are presented for one mix parameter at a time in subsequent subsections. Please note that for brevity only select results are presented herein. The database and analysis files that are accompanying this report contain the full set of results.

2.3.2.1.1 <u>Asphalt Film Thickness (AFT)</u>

The asphalt film thickness of mixes was compared against ITS (dry and wet) and TSR. The range of values for adjusted AFT was between 4 and 12 microns, and for those only based on P_{be} between 2 and 7 microns. A plot of the data points for AFT Adjusted and ITS (wet) is shown in Figure 2.10: Asphalt Film Thickness vs. Wet Strength. The plot and the statistical analysis showed that no clear relationship exists between adjusted AFT and wet ITS. This poor relationship between the AFT mix parameter (both P_{be} and adjusted) was also evident in the analysis against dry ITS and TSR. This poor relationship corresponds to AFT not having significance on the ITS or TSR.

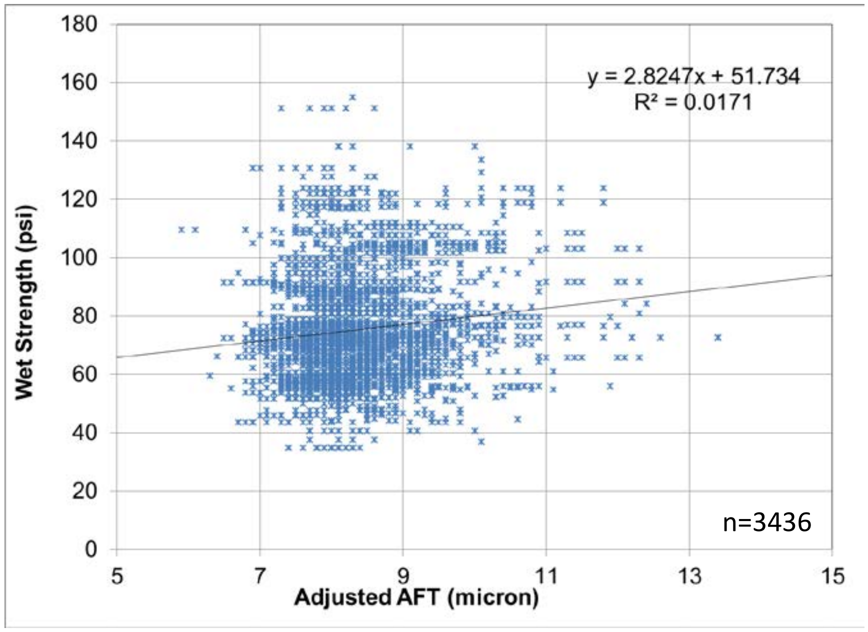

Figure 2.10: Asphalt Film Thickness vs. Wet Strength

2.3.2.1.2 Design Air Void Level

Design air void levels that were extracted from the database represented three distinct values of 3.0%, 3.5%, and 4.0%. Initial graphing of the design air void level against ITS (dry) showed that a better representation of these results needed to be conducted, as seen in Figure 2.11.

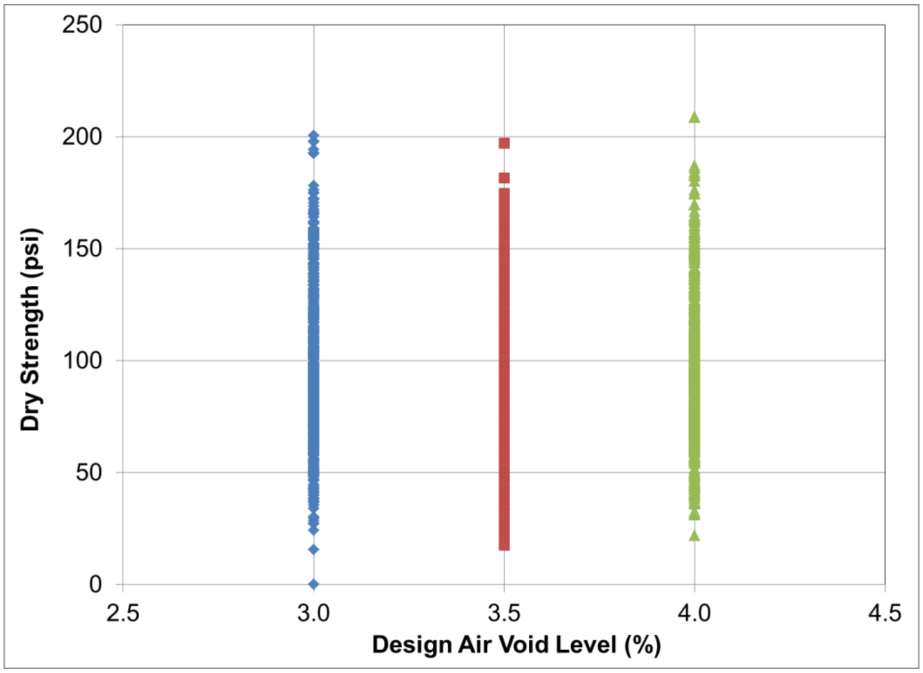

Figure 2.11: Initial plotting of ITS (dry) versus design air void level

A better representation was done by calculating the normalized frequencies of ITS and TSR intervals at 10 psi and 10% increments for each design air void level. Each frequency level represents the percent of mixes that were present for the given interval at the air void level. The normalized frequency plot for ITS (dry) and design air voids is present is Figure 2.12.

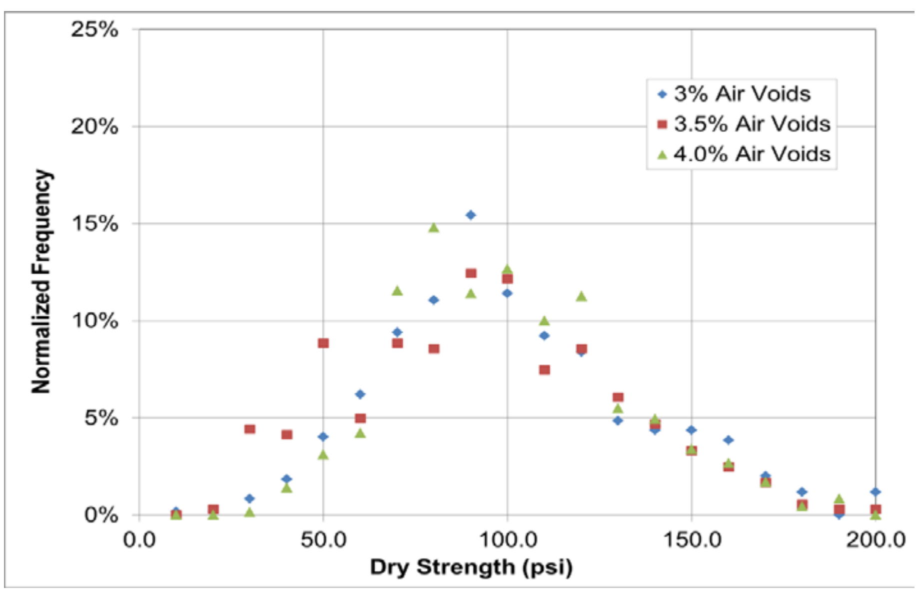

Figure 2.12: Normalized frequency plot of ITS (dry) for each design air void level

Table 2.6: ITS (dry) and design air void level statistics

ITS (dry) (psi)			
	3.0% Air Voids	3.5% Air Voids	4.0 % Air Voids
Median	90.6	88.0	92.7
Average	95.8	88.6	96.2
Standard Deviation	34.4	35.6	30.8

These normalized frequency plots allowed a visual representation of the average ITS or TSR value for each air void level. They also show if the spread in data and the mean values of ITS or TSR varied with the design air void levels. The median, average and standard deviation of each data set was also calculated and shown in Table 2.6.

No noticeable trend is seen in the plot (Figure 2.12) and the statistical information (Table 2.6) also reaffirms this claim. This indicates that the correlation between design air void level and ITS (dry) is poor. Similar results were also seen during analysis of design air level against ITS (wet) and TSR.

2.3.2.1.3 Measured Air Void Level

Actual air void level measurements were compared against ITS and TSR. The measured air void content of mixes ranged from values of 1-6 %. A plot of measured air voids and ITS (dry) is presented in Figure 2.13. Little correlation can be seen with a minor trend of increasing strength values with increasing air void level. The air void level and TSR show a very weak correlation between the two as well (c.f. Figure 2.14), with a trend of decreasing TSR with increasing air voids. This trend is expected as asphalt mixes with higher air voids tend to show decrease in strength after undergoing moisture conditioning. A similar relationship was also seen between the measured air voids and ITS (wet).

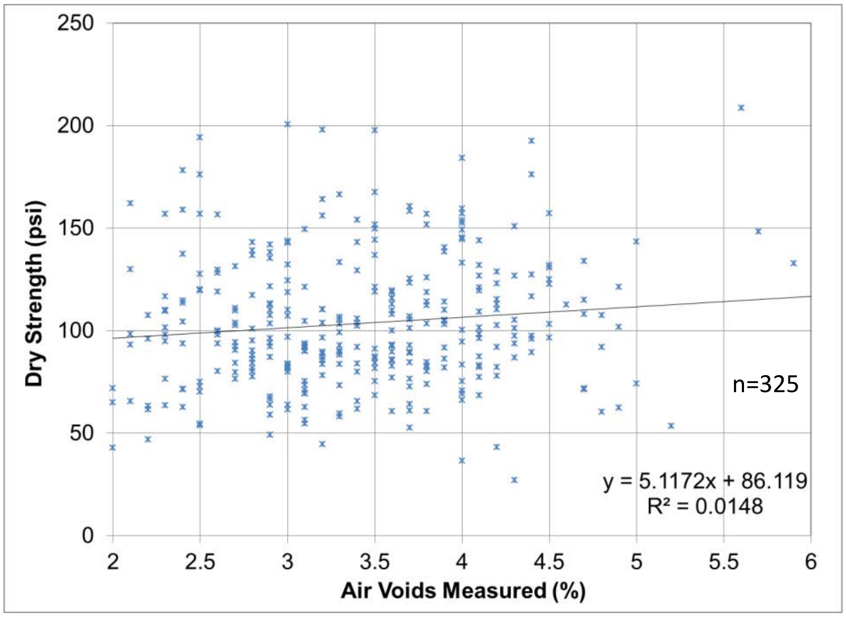

Figure 2.13: Measured air void level versus ITS (dry)

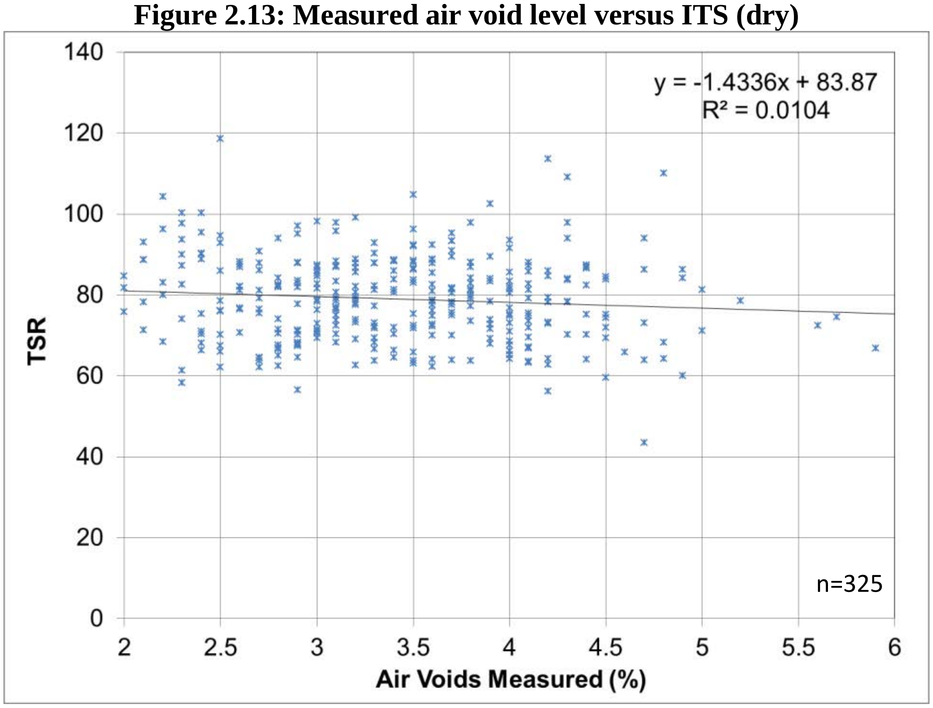

Figure 2.14: Measured air void level versus TSR

2.3.2.1.4 Mix Size (Nominal Maximum Aggregate Size)

The Nominal Maximum Aggregate Size (NMAS) represents the largest sieve size that retains less than 10% aggregate by weight. The NMAS of asphalt mixes were compared against ITS and TSR. The comparisons were conducted for 3/4 in., 1/2 in., 3/8 in., and 0.187 in. (#4) sized mixes. The data was analyzed using a normalized frequency, as it was discretized in four mix sizes. The normalized plots ITS (dry) for each of the four NMAS are presented in Figure 2.15.

The basic data statistics are shown in Table 2.7. The shaded columns of results represent NMAS values that contained very few data points. Due to a small representation of data for these NMAS values, the focus of results is on NMAS values of 1/2 in. and 3/8 in. The 1/2 in. NMAS did show a slightly higher value for average and median dry strength as compared to the 3/8 in. This very slight correlation of mix size to strength was also evident with wet strength. The TSR analysis showed no relationship between mix size and TSR value. The wet strength and TSR normalized plots are shown in Figure 2.16.

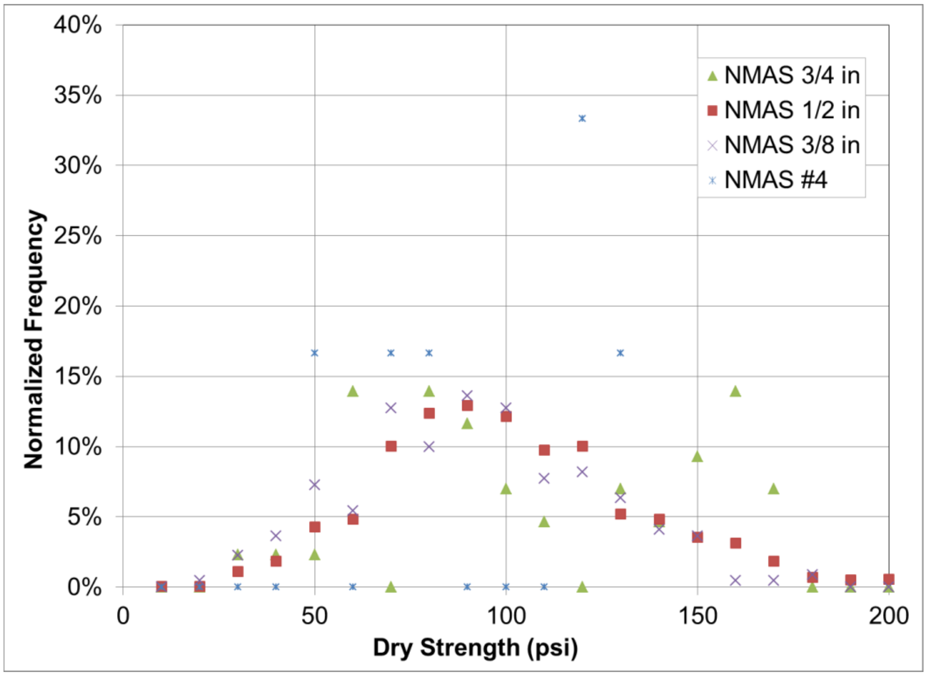

Figure 2.15: Normalized frequency plot of ITS (dry) for various NMAS

Table 2.7: ITS and TSR statistics for various mix sizes (NMAS)

ITS (dry) (psi)				
Mix Size	3/4 in.	1/2 in.	3/8 in.	#4
Median	93.6	91.9	84.3	93.7
Average	103.5	95.4	87.0	91.3
Standard Deviation	42.4	33.1	31.2	26.1
ITS (wet) (psi)				
Mix Size	3/4 in.	1/2 in.	3/8 in.	#4
Median	74.9	72.2	66.9	73.5
Average	76.9	74.8	67.9	76.2
Standard Deviation	25.7	24.5	22.7	23.1
TSR (%)				
Mix Size	3/4 in.	1/2 in.	3/8 in.	#4
Median	77.8	79.2	79.8	84.8
Average	77.9	79.6	79.5	85.1
Standard Deviation	12.1	10.4	11.1	10.9

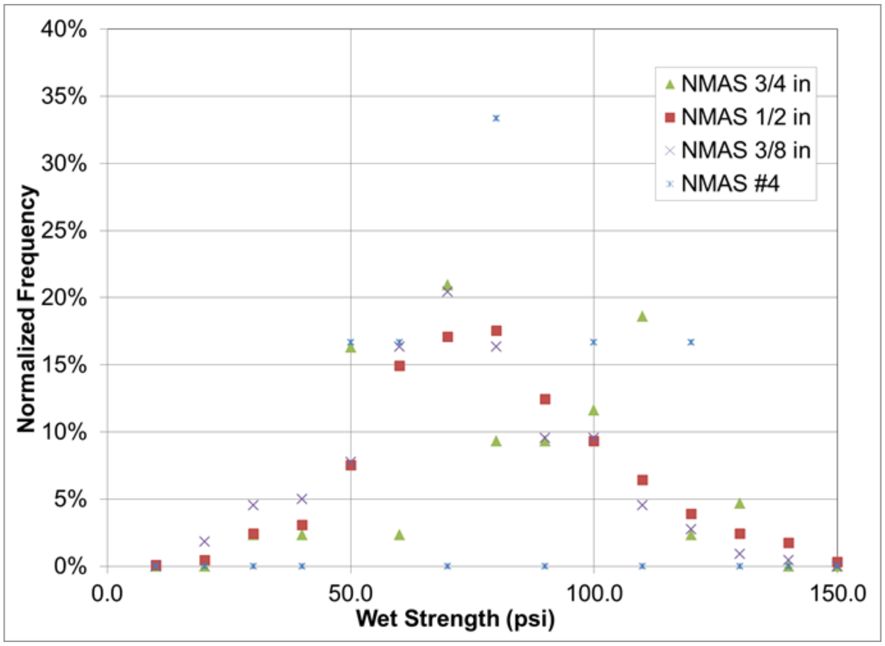

Figure 2.16: Normalized frequency plot of ITS (wet) for various NMAS

2.3.2.1.5 Asphalt Binder Content (Percent Binder)

The amount of asphalt binder present in mixes, measured using both ignition and extraction methods, was statistically evaluated. As stated previously, ignition and extraction refer to different laboratory methods used to determine the asphalt content of mixes. Figure 2.17 shows the spread of data for asphalt binder content versus ITS (dry). No discernible correlation between asphalt binder content and ITS was observed A very slight increase of ITS can be seen from a linear fit.

When percent binder content was compared to ITS (wet) and TSR, the trend of minimal to no correlation was seen. For TSR, the values stayed fairly constant between the different binder percentages. The strengths slightly increased with increasing percent binder amounts, but this effect was minimal. The analysis showed the same results for percent binder content determined using chemical extraction.

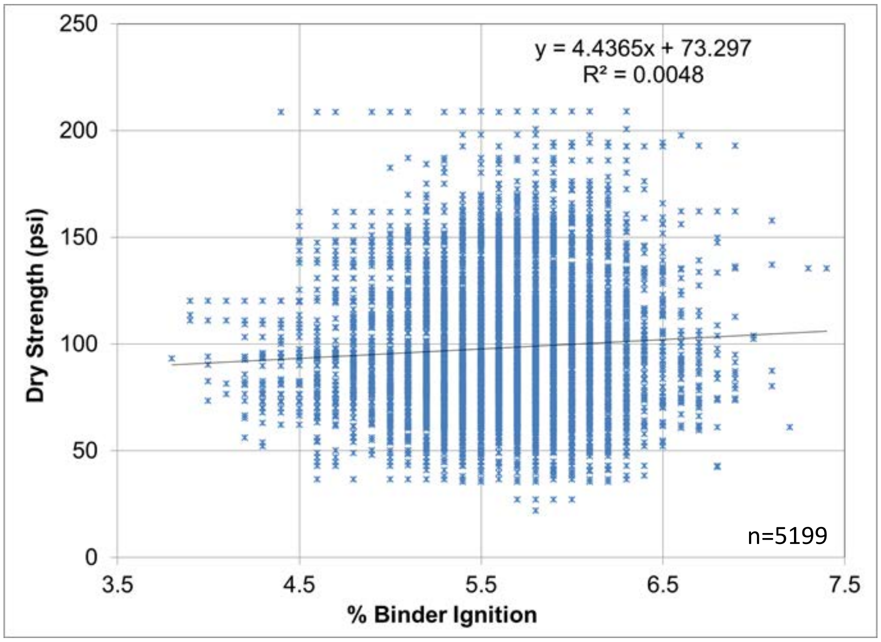

Figure 2.17: Percent asphalt binder content (ignition) versus ITS (dry)

2.3.2.1.6 <u>Asphalt Binder Grade (PG Grade)</u>

The PG Grade of mixes found within the database included eight different grades. These eight grades are shown along with the percent of mixes that used a given type of grade in Table 2.8. For the entire amount of PG grade data in the database, a significant amount exists for PG 58-34 and PG 58-28. This can be attributed to these being the most widely used asphalt binder grades in Minnesota. Binder grades of PG 58-40, PG 64-22, and PG 70-34 represented less than 1% of total data extracted from the database.

Table 2.8: Distribution of data of each binder grade

PG Grade	Percentage of Mixes in Database
PG 58-28	63.40%
PG 58-34	20.50%
PG 58-40	0.20%
PG 64-22	0.50%
PG 64-28	9.80%
PG 64-34	4.40%
PG 70-28	1.00%
PG 70-34	0.20%

The normalized frequencies of ITS (dry) for each binder grade is plotted in Figure 2.18. The basic statistical data for the ITS (dry and wet) and TSR for each binder grade is presented in Table 2.9. The grayed out columns represent binder grades with limited amount of data and thus it may not have a representative number of mixes to draw a reliable conclusion. The comparison of average ITS values constantly show that softer binder grades yield lower strength. This is expected as the ITS in AASHTO T-283 test is measured at 25 °C, where the mechanical behavior of mix is driven significantly by the binder behavior. The TSR showed little dependence on binder grade. Binders with greater spread in high and low temperature grades showed slightly higher values.

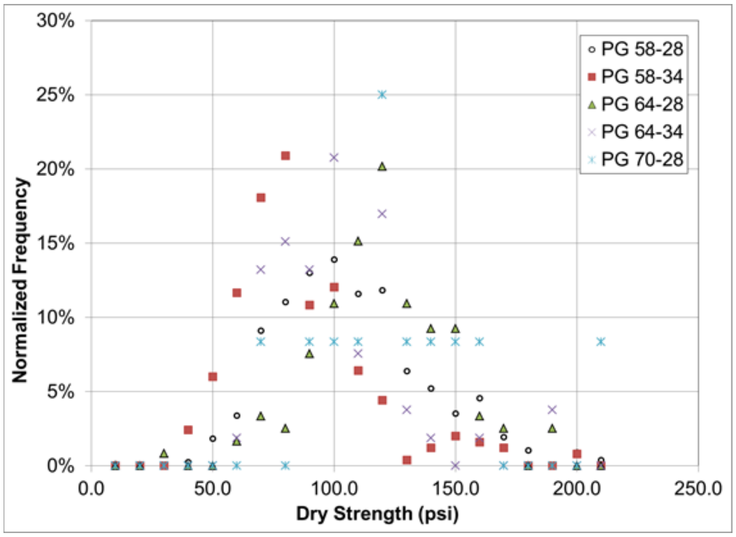

Figure 2.18: Normalized frequency plot of ITS (dry) for various asphalt binder grades (PG)

Table 2.9: ITS and TSR statistics for various asphalt binder grades (PG)

ITS (dry) (psi)								
Grade	PG 58-28	PG 58-34	PG 58-40	PG 64-22	PG 64-28	PG 64-34	PG 70-28	PG 70-34
Median	97.20	75.90	85.90	117.10	114.50	92.00	116.05	103.70
Average	101.84	81.08	85.90	119.18	115.16	96.50	122.23	103.70
Standard Deviation	31.08	28.00	---	---	27.38	26.82	36.30	---
ITS (wet) (psi)								
Grade	PG 58-28	PG 58-34	PG 58-40	PG 64-22	PG 64-28	PG 64-34	PG 70-28	PG 70-34
Median	76.00	62.60	66.75	103.80	91.60	78.00	101.40	91.35
Average	78.58	65.42	66.75	104.48	94.03	79.17	101.08	91.35
Standard Deviation	22.26	18.32	---	---	23.05	19.87	23.26	---
TSR (%)								
Grade	PG 58-28	PG 58-34	PG 58-40	PG 64-22	PG 64-28	PG 64-34	PG 70-28	PG 70-34
Median	78.00	81.90	77.90	85.25	82.00	83.50	83.70	87.85
Average	78.23	82.68	77.90	87.48	82.19	82.89	84.17	87.85
Standard Deviation	9.57	10.58	---	---	9.56	8.88	8.12	---

The asphalt binder grade data can be further analyzed with focus only on the low temperature grade of the binder, referred to as “PGLT”. The main reason for evaluating binder grade data in context of PGLT is to focus on the thermal cracking behavior, which is the focus of this research. The distribution of the PGLT amongst the mixes present in database is tabulated in Table 2.10.

Table 2.10: Distribution of data for PGLT

PGLT (°C)	Percentage of Mixes in Database
-22	0.5%
-28	73.1%
-34	26.2%
-40	0.2%

The ITS and TSR data corresponding to each PGLT was converted to normalized frequencies and plotted (Figure 2.19). A significant increase in strength for mixes with PGLT of -34 °C to -28 °C was observed, this is evident from the frequency plot as well as the average ITS values shown in . As stated previously, this decrease in strength for mixes with PGLT -34 °C binders over -28 °C binders is partially due to testing temperature associated with AASHTO T 283 specification.

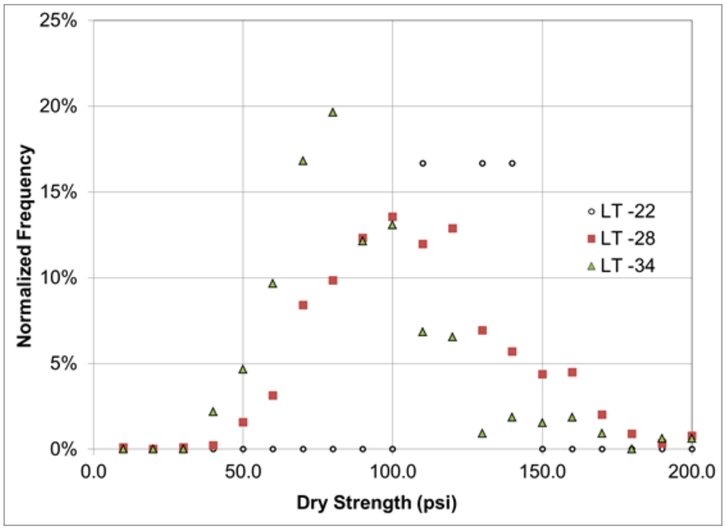

Figure 2.19: Normalized frequency plot of ITS (dry) for various low temperature binder grades (PGLT)

Table 2.11: ITS (dry) for various low temperature asphalt binder grades (PGLT)

ITS (dry) (psi)				
PGLT	-22 °C	-28 °C	-34 °C	-40 °C
Median	117.1	100.8	79.2	85.9
Average	119.2	103.8	84.2	85.9
Standard Deviation	9.2	31.1	28.5	28.1

An alternative for evaluation of dependence of ITS and TSR on the type of asphalt binder is to look at data from the perspective of the spread in the binder grade. The PG spread of binder is essentially the difference between the high and low temperature grade of the binder. For the mixes present in the database the spreads of 86, 92, 98 and 104°C were found. The distribution of the data falling under these spreads as well as the asphalt binders that provide these spreads are listed in Table 2.12. Due to use of PG 58-28 and PG 58-34 being primary asphalt grades for large amount of asphalt mixes in Minnesota, significant amount of data fell in 86 and 92 °C spread category.

Table 2.12: Distribution of data for PG spread

PG Spread (°C)	PG Grades with listed Spread	Percentage of Data
86	PG 58-28, PG 64-22	64.2%
92	PG 58-34, PG 64-28	30.0%
98	PG 58-40, PG 64-34, PG 70-28	5.6%
104	PG 70-34	0.2%

The normalized frequency plots for the ITS (dry) and PG spreads of 86, 92 and 98 °C are presented in Figure 2.20. The statistics for the ITS (dry) for mixes in each PG spread category is tabulated (Table 2.13). The data shows that higher ITS values are present for mixes with PG spread of 86 and 98 °C as compared to 92 °C. The primary amount of mixes with PG spread of 98 °C represent binders with PG 64-34 and PG 70-28 grades, thus higher ITS values are expected. The PG spread did not show any discernible trends with TSR.

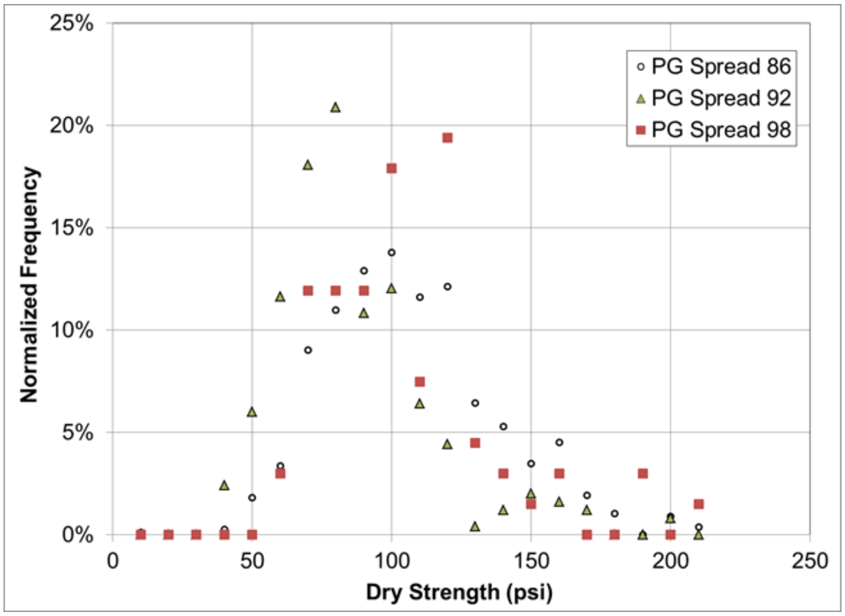

Figure 2.20: Normalized frequency plot of ITS (dry) for various spreads in binder grade (PG Spread)

Table 2.13: ITS (dry) for various spreads in asphalt binder grades (PG Spread)

ITS (dry) (psi)				
PG Spread	86 °C	92 °C	98 °C	104 °C
Median	97.90	86.95	97.40	103.70
Average	101.98	91.65	100.79	103.70
Standard Deviation	31.01	32.04	30.03	---

From analyzing PG Grade, PGLT, and PG Spread data, it can be concluded that there was a relationship between the binder type and ITS. This trend was especially evident binders with low temperature PG of -28 and -34 °C. This dependence is reflection of the test temperature of the AASHTO T 283 procedure. This test is run at 25°C, and it is run at this temperature regardless of what PG Grade is used in the specimen. This temperature does not accurately reflect the low temperatures pavements in Minnesota and other cold regions are subjected to during the winter months. The test temperature needs to be much lower in order to replicate what the mix will be subjected to when placed in the field. To replicate how the specimens will act and behave in environments for which they are rated for, they should be tested at temperatures for which they are designed. A mix using a PG Grade with a low temp of -34 °C should be tested around that temperature range rather than at 25°C to model in place conditions.

2.3.2.1.7 Design Traffic Level

The database included mixes designed at traffic levels of 2, 3, 4, and 5. These traffic levels correspond to 20 year design ESALs as described in MnDOT 2360 specifications. The distribution of data extracted from the database pertaining to traffic level can be seen in Table 2.14. A large portion of the data corresponds to traffic levels of 2 and 3. The normal frequency plots of ITS (dry) for various traffic levels are presented in Figure 2.21. It can be seen that as the design traffic level increases the data generally shifts towards greater ITS. The basic statistical information for ITS (dry) data at each traffic level, shown in , also indicates the same.

Table 2.14: Distribution of data for design traffic levels

Traffic Level	Percentage of Data
2	37.2%
3	45.0%
4	16.3%
5	1.6%

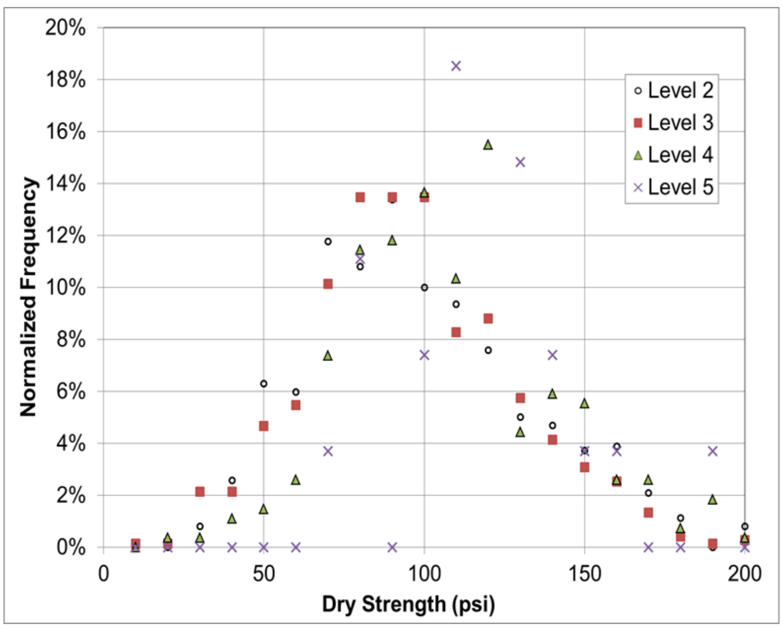

Figure 2.21: Normalized frequency plot of ITS (dry) for various design traffic levels

Table 2.15: ITS (dry) for various design traffic levels

ITS (dry) (psi)				
Design Traffic Level	2 (< 1 million ESAL)	3 (1-3million ESAL)	4 (3-10 million ESAL)	5 (10–30 million ESAL)
Median	88.6	88.75	99.8	112.7
Average	93.5	91.2	103.1	117.8
Standard Deviation	34.7	31.9	31.8	31.4

The increase in ITS with increasing design traffic levels can be attributed to the greater amounts of crushed aggregate requirement at high traffic levels. For example, the MnDOT 3139 specifications require that mixes produced for traffic level 4 have 85% coarse aggregate with at least one crushed face and 8% with two or more versus a level 3 mix only requires 55% coarse aggregates to have at least one crushed face. With the ITS testing conducted at 25 °C, the aggregate shape plays an important role in the measured strength. Mixes with greater amount of crushed aggregates have capability to carry higher loads prior to failure.

The statistical measures for the TSR data at various design traffic levels is tabulated in Table 2.16. The average TSR values for level 2 and 3 mixes are very close to each other. The increase in TSR values for level 4 and 5 is evident form the data and follows the higher requirement for TSR as per the MnDOT 2360 specifications.

Table 2.16: TSR for various design traffic levels

TSR (%)				
Design Traffic Level	2 (< 1 million ESAL)	3 (1-3million ESAL)	4 (3-10 million ESAL)	5 (10–30 million ESAL)
Median	78.4	78.9	81.5	83.8
Average	78.9	79.28	81.5	85.3
Standard Deviation	11.1	10.48	9.0	9.3

2.3.2.1.8 Voids in Mineral Aggregates (VMA)

The VMA calculated using the chemical extraction and ignition oven procedures were used for analysis. The data scatter for ITS (dry) and VMA (extraction) is presented in Figure 2.22. The data analysis demonstrated that for VMA calculated using either methods of chemical extraction or ignition oven, the ITS (dry and wet) and TSR showed minimal to no correlation.

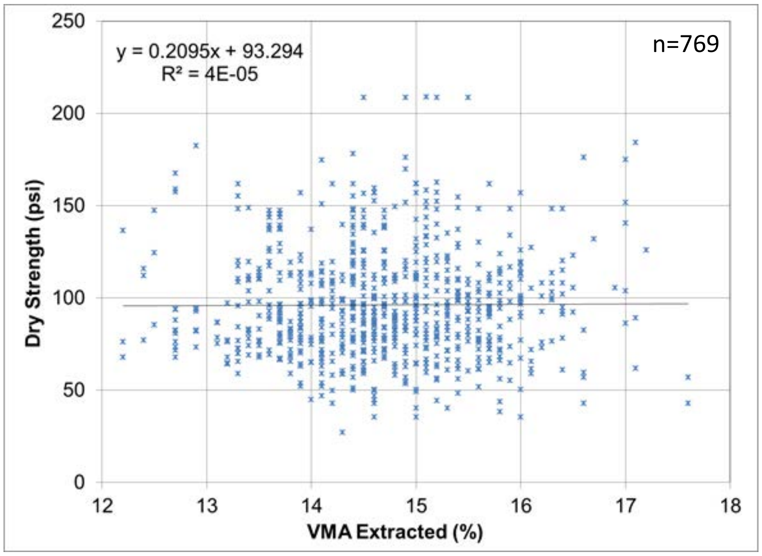

Figure 2.22: Voids in mineral aggregate (VMA chemical extraction method) versus ITS (dry)

2.3.2.1.9 Voids Filled with Asphalt (VFA)

The VFA values were compared with ITS (dry and wet) and TSR. Like VMA, the VFA values determined using chemical extraction and ignition oven methods were available in the database. The data scatter for ITS (dry) against VFA (ignition method) is plotted in Figure 2.23. The data analysis showed that ITS and TSR did not correlated with the VFA from either methods (extraction and ignition).

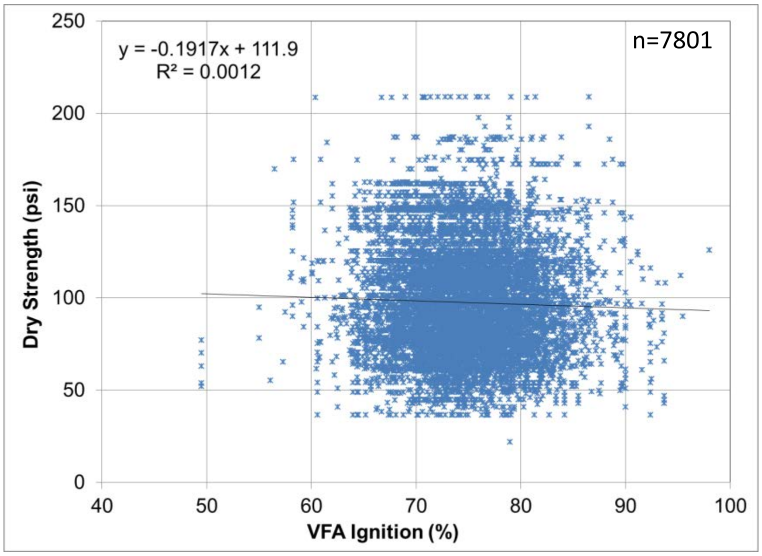

Figure 2.23: Voids filled with asphalt (VFA ignition oven method) versus ITS (dry)

2.3.2.1.10 Summary of Single Variable Analysis

The results from statistical testing of the ITS (dry) dependence on mix design parameters is summarized in Table 2.17. The only mix design parameters that caused ITS (dry) to have statistically significant dependence are the asphalt binder grade, air void level, mix size and design traffic level. Analysis showed that a change in PG Grade, in terms of low temperature of -34 to -28 °C, resulted in an increase in ITS. This may be attributed to the temperature at which the strength testing is conducted for AASHTO T 283 procedure. The traffic level correlating to mix strength can be attributed to the amount of fractured or crushed aggregate required in mixes depending on design traffic level. The increasing air void level showed very slight increase in the ITS (dry) and the increasing mix size showed minor increase in ITS (dry) as well. The ITS (wet) values were also compared against the mix design parameters to determine if it depended on them in a statistically significant manner. The results are presented in Table 2.18. In general, the results follow similar trends as discussed for ITS (dry), the only difference is that ITS (wet) did not demonstrate dependence on the actual air void level.

Table 2.17: Significance of mix design parameters on ITS (dry)

Mix Parameters	p-value	ITS (dry) is related to mix parameters?
AFT Adjusted	0.2121	no
AFT Pbe	0.8865	no
Percent Binder (Chemical extraction)	0.2559	no
Percent Binder (Ignition oven)	0.2453	no
VMA (Chemical extraction)	0.1633	no
VMA (Ignition oven)	0.0313	yes (weak)
PG Grade	< 0.0001	yes
VFA (Chemical extraction)	0.0669	no
VFA (Ignition oven)	0.7937	no
Design Air Void Level	0.0009	yes
Actual Air Void Level	0.0081	yes
Mix Size (NMAS)	0.0014	yes
Design Traffic Level	< 0.0001	yes

Table 2.18: Significance of mix design parameters on ITS (wet)

Mix Parameters	p-value	ITS (wet) is related to mix parameters?
AFT Adjusted	0.0969	no
AFT Pbe	0.8953	no
Percent Binder (Chemical extraction)	0.0536	no
Percent Binder (Ignition oven)	0.2188	no
VMA (Chemical extraction)	0.6859	no
VMA (Ignition oven)	0.0709	no
PG Grade	< 0.0001	yes
VFA (Chemical extraction)	0.1395	no
VFA (Ignition oven)	0.5256	no
Design Air Void Level	< 0.0001	yes
Actual Air Void Level	0.1716	no
Mix Size (NMAS)	0.0013	yes
Design Traffic Level	< 0.0001	yes

The TSR measurements in the database were also analyzed to determine if there was statistically significant dependence of TSR on various mix design parameters. The summary of this analysis is presented in Table 2.19. The results show that only asphalt binder grade (PG), design air void level, and design traffic level were significant variables affecting the TSR of mixes. The results from the single variable analysis were used to decide on groupings of mix parameters to be analyzed during the multiple variable analyses, which are discussed in the next section.

Table 2.19: Significance of mix design parameters on TSR

Mix Parameters	p-value	TSR is related to mix parameters?
AFT Adjusted	0.1822	no
AFT Pbe	0.5294	no
Percent Binder (Chemical extraction)	0.2549	no
Percent Binder (Ignition oven)	0.0522	no
VMA (Chemical extraction)	0.8959	no
VMA (Ignition oven)	0.4088	no
PG Grade	< 0.0001	yes
VFA (Chemical extraction)	0.5668	no
VFA (Ignition oven)	0.1529	no
Design Air Void Level	0.0007	yes
Actual Air Void Level	0.3137	no
Mix Size (NMAS)	0.4261	no
Design Traffic Level	0.0002	yes

2.3.2.2 Multiple Variable Analysis

The multiple variable analysis was the next step in evaluating the effects of mix design parameters on ITS and TSR. In the single variable analysis, only one mix parameter was compared to the ITS and TSR values. The multiple variable analysis grouped mix parameters together to quantify their combined effect on strength and TSR. It is important to conduct this type of analysis, since while independently two parameters may not show significant effect on ITS in a combined manner they might be significant. For example, the VMA of the mix may not be significantly affecting the ITS when looked independently, but combined effects of VMA and asphalt content may be significant.

The groupings of mix parameters were done based on the results obtained from the single variable analysis. These groupings are illustrated in Figure 2.24. The mix parameter that was found to have dependence to mix strength in the single variable analysis was the PG Grade. Due to these parameters already being known to influence the mix strength, they were paired with other mix parameters. The PG Grade of different mixes was combined with VMA, Asphalt Content, and AFT for multiple variable analyses. AFT and VMA were also paired with asphalt content to quantify their combined effects on mix strength and TSR.

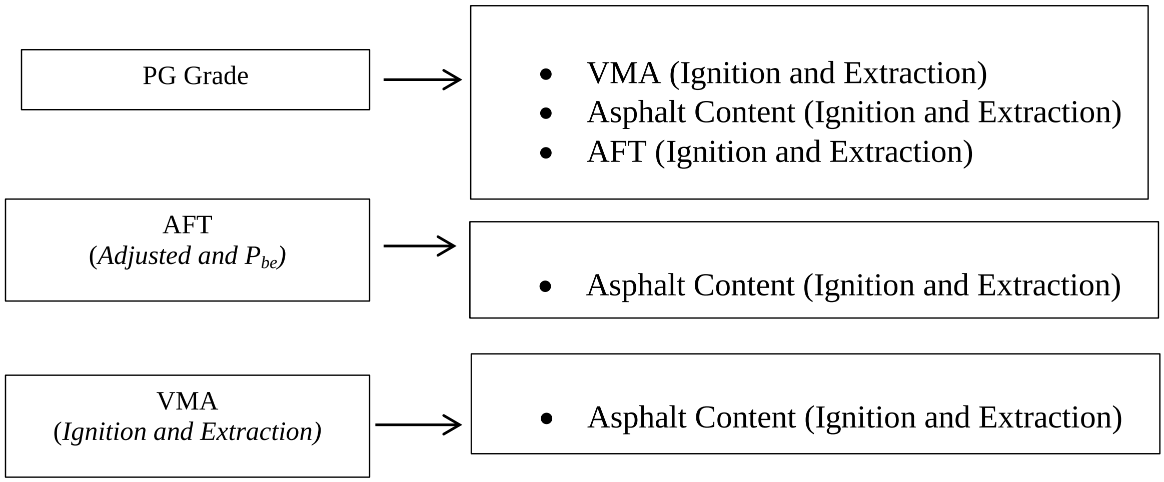

Figure 2.24: Groupings for the multiple variable analysis of mix parameters

The combined effects of the pairings on ITS (dry and wet) and TSR was determined using the least-squares mean analysis using the statistical analysis software SAS. The lists of paired parameters were first obtained from the comprehensive database. To refine the results from the statistical analysis, bounds were put on the different mix parameter values. Character values were given to define these bounds. The bounds put on the mix parameter values as well as the characters assigned to each bound can be seen in Figure 2.25.

VMA (%)	Character
< 11	A
11-12	B
12-13	C
13-14	D
14-15	E
> 15	F

% Binder	Character
< 3.5	A
3.5-4.0	B
4.0-4.5	C
4.5-5.0	D
5.0-5.5	E
5.5-6.0	F
> 6.0	G

AFT (microns)	Character
< 6.0	A
6.0-7.0	B
7.0-7.5	C
7.5-8.0	D
8.0-8.5	E
8.5-9.0	F
9.0-10	G
> 10	H

PG Grade	Character
52-34	A
58-28	B
58-34	C
64-28	E
64-34	F
70-28	H
70-34	I
64-22	L

PG LT	PG Grade	Character
-28	58-28, 64-28, 70-28,	A
-34	52-34, 58-34, 64-34, 70-34	B
-22	64-22	C

PG Spread	PG Grade	Character
86	52-34, 58-28, 64-22	A
92	58-34, 64-28	B
98	64-34, 70-28	C
104	70-34	D

Figure 2.25: Character keys for the bounds of various asphalt mix parameters

The results of the least-square means analysis are presented in concise summary tables. The dependence of ITS (dry) on grouped mix parameters are shown in Table 2.20. The results for similar set of analyses with the ITS (wet) is presented in Table 2.21. The results are very similar to those obtained for ITS (dry).

Table 2.20: Dependence of ITS (dry) on grouping of mix parameters

Mix Parameters	p-value	ITS (dry) is related to mix parameters?
PG Grade and VMA Extracted	0.0977	no
PG Grade and VMA Ignition	0.0535	no
PG Grade and Percent Binder Extracted	0.1292	no
PG Grade and Percent Binder Ignition	<0.0001	yes
PG Grade and AFT Adjusted	<0.0001	yes
AFT Adjusted and Percent Binder Ignition	<0.0001	yes
AFT Adjusted and Percent Binder Extracted	0.4029	no
AFT P_{be} and Percent Binder Extracted	-	no
AFT P_{be} and Percent Binder Ignition	-	no
VMA Extracted and Percent Binder Extracted	0.0024	yes
VMA Extracted and Percent Binder Ignition	0.4082	no
VMA Ignition and Percent Binder Extracted	0.0678	no
VMA Ignition and Percent Binder Ignition	<0.0001	yes

Table 2.21: Dependence of ITS (wet) on grouping mix parameters

Mix Parameters	p-value	ITS (wet) is related to mix parameters?
PG Grade and VMA Extracted	0.1721	no
PG Grade and VMA Ignition	0.3070	no
PG Grade and Percent Binder Extracted	0.2474	no
PG Grade and Percent Binder Ignition	<0.0001	yes
PG Grade and AFT Adjusted	0.0040	yes
AFT Adjusted and Percent Binder Ignition	<0.0001	yes
AFT Adjusted and Percent Binder Extracted	0.0731	no
AFT P_{be} and Percent Binder Extracted	-	no
AFT P_{be} and Percent Binder Ignition	-	no
VMA Extracted and Percent Binder Extracted	0.0286	yes
VMA Extracted and Percent Binder Ignition	0.7975	no
VMA Ignition and Percent Binder Extracted	0.1139	no
VMA Ignition and Percent Binder Ignition	0.0020	yes

The analysis shows that the ITS on the mix is strongly dependent on pairings of PG grade and asphalt binder content (ignition method), PG grade and adjusted AFT, adjusted AFT and percent binder content (ignition method), and VMA (ignition method) and percent binder content (ignition method). The ITS also showed dependence on the pairing of VMA (extraction method) and percent binder content (extraction method), but this dependence is weaker than the ones listed before.

The mix parameter pairings involving percent binder content using chemical extraction yielded results of no correlation to ITS, with exception to the pairing with VMA (extraction method). In contrast, pairings including percent binder content determined using ignition oven, with exception to being paired with VMA (extraction method), yielded correlation to ITS. This suggests that the method of determining the percent binder content in asphalt mixes can bias the ITS of mix and require further investigation.

Since the asphalt mix design procedures rely heavily on use of volumetric quantities such as AFT or VMA, it was expected that through groupings with either binder content or PG grade the ITS will show dependence on these. However the results indicate a non-consistent dependence of ITS on either AFT or VMA. This result indicates that if the ITS was to be used as performance measure, the presence of either AFT or VMA requirements in specification cannot substitute the need for measuring ITS through laboratory testing.

The TSR data was also analyzed to determine its dependence on grouping of mix variables. The results are presented in Table 2.22. The analysis resulted in only one strong dependence, which is for the grouped pairing of PG grade and percent binder content (ignition method). Other dependencies varied from intermediate to weak. Once again, no clear dependence for the volumetric measures (AFT and VMA) were seen when they were grouped with PG grade or asphalt content.

Table 2.22: Dependence of TSR on grouping of mix parameters

Mix Parameters	p-value	TSR is related to mix parameters?
PG Grade and VMA Extracted	0.1705	no
PG Grade and VMA Ignition	0.0145	yes
PG Grade and Percent Binder Extracted	0.7439	no
PG Grade and Percent Binder Ignition	<0.0001	yes
PG Grade and AFT Adjusted	0.6000	no
AFT Adjusted and Percent Binder Ignition	0.0020	yes
AFT Adjusted and Percent Binder Extracted	0.0099	yes
AFT P_{be} and Percent Binder Extracted	-	no
AFT P_{be} and Percent Binder Ignition	-	no
VMA Extracted and Percent Binder Extracted	0.0042	yes
VMA Extracted and Percent Binder Ignition	0.0551	no
VMA Ignition and Percent Binder Extracted	0.0118	yes
VMA Ignition and Percent Binder Ignition	0.0020	yes

2.3.2.3 Summary of Analysis to Evaluate Effects of Mix Design Parameters on ITS and TSR

The analysis of the ITS and TSR data with respect to various asphalt mix design parameters provided insight into dependence of these mechanical properties on the mix design. The analysis yielded following findings:

- Most asphalt mix volumetric parameters (such as, AFT, VMA, VFA and air void level) show minimal to no influence on the ITS. The TSR of mix deteriorates slightly with increase in air void level.
- Increase in the asphalt binder content (% AC) leads to reduction in ITS of the mix. This indicates that the ITS, as determined using AASHTO T-283 procedure which is at 25 °C, may be decreasing as the mixes become more ductile in nature.
- The ITS of mix decreases with use of softer asphalt binder grade such as, PG 58-34 as compared to PG 58-28. The low temperature binder grade (PGLT) has a significant effect on ITS. The drop in ITS with use of softer binder also supports the hypothesis that ductile mixes will have lower ITS.

- The ITS values for traffic level 4 and 5 mixes are significantly higher than level 2 and 3 mixes. The increase in ITS is anticipated due to increase amount of crushed aggregates in high traffic level mixes.

The minimal dependence of ITS on mix volumetrics and the decreasing trend in ITS values with use of softer binder grades and higher binder amounts reduce the confidence for its use as a pavement cracking performance indicator. However, it is important to directly evaluate the correlation between actual field cracking performance of asphalt pavements in Minnesota against the corresponding ITS values of the mixes before drawing the final conclusion on the topic. The next section presents the analysis between field cracking measures and ITS (and TSR) to determine if these lab measured parameters can be used to predict the field performance and in-turn can be used as laboratory performance test.

2.3.3 Correlation between Field Cracking Measures and ITS

The contents of this section include description of statistical analysis that was conducted for investigating the relationship between ITS (dry and wet) and TSR against field cracking performance. The field cracking performance was expressed in the form of twelve different measures of transverse and longitudinal cracking. For transverse cracking these measures are: Maximum Total Transverse Cracking Amount (MTCTotal), Maximum Total Weighted Transverse Cracking Amount (MTCWeighted), Maximum Total Transverse Cracking Rate (MTCRTotal), Maximum Total Weighted Transverse Cracking Rate (MTCRWeighted), Average Total Transverse Cracking Rate (ATCTotal), and Average Weighted Total Transverse Cracking Rate (ATCWeighted). Similar measures were also used for longitudinal cracking. The detailed description of these measures and their calculation are discussed in Chapter 2 of this report (refer to Table 2.2).

The analysis and results presentation for this section is divided in two potions. The first subsection discusses the maximum total and weighted cracking amounts and rates whereas the second subsection focuses on the average cracking rates.

2.3.3.1 Effects of ITS and TSR on Maximum Cracking Amounts and Rates

2.3.3.1.1 <u>ITS (dry)</u>

The linear regression analysis of the maximum transverse and longitudinal cracking amounts and rates provided insight into relationships between the ITS (dry) of asphalt mixes and the field cracking performance. The results from this analysis are presented in the form of summary table as well as the scatter plots showing the linear fitting lines and quality of fit (R^2). The summary of analysis is provided as . The results show that ITS (dry) of the mix has an effect on all maximum transverse and longitudinal cracking amounts and rates except the maximum total longitudinal cracking rate. It should be noted that the primary cracking parameter of interest in this study is the maximum weighted transverse cracking amount (MTCWeighted), as it is expected to be the most encompassing measure that is closely related to the pavement durability and ride quality.

Table 2.23: Effect of ITS (dry) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to ITS (dry)?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	0.0012	yes
MLCRTotal	0.4671	no
MLCRWeighted	0.0023	yes

In order to further analyze the effects of ITS (dry) on the pavement transverse cracking, the complete dataset for ITS (dry) is plotted against the MTCWeighted in Figure 2.26. Few observations can be drawn from this plot. First, the data appears to have quite a significant amount of spread; this is also evident from low R2 for the fitted linear curve. Secondly, it can also be seen that the variation in the MTCWeighted is relatively small over large range of ITS (dry) values. Finally, it can also be seen that the data trend from the linear fit indicates that the mixes with greater ITS (dry) undergo higher amount of transverse cracking. The ITS (dry) is plotted against the MTCTotal, which represents sum total of low, medium and high severity transverse cracking amounts (c.f. Figure 2.27). Similar observations can be made with this plot as the previous one. The data scatter and linear regression fits for the longitudinal cracking measures are similar in nature to those for transverse cracking.

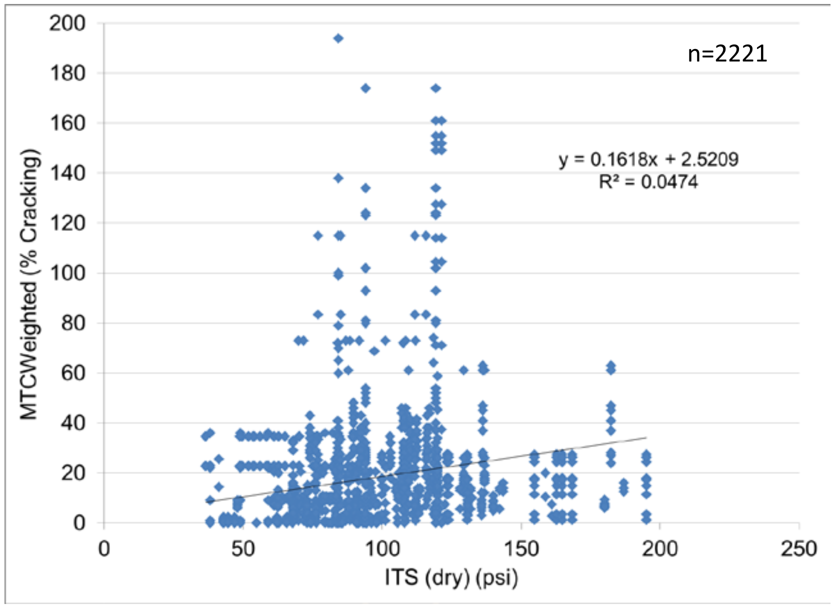

Figure 2.26: ITS (dry) and maximum total weighted transverse cracking (MTCWeighted)

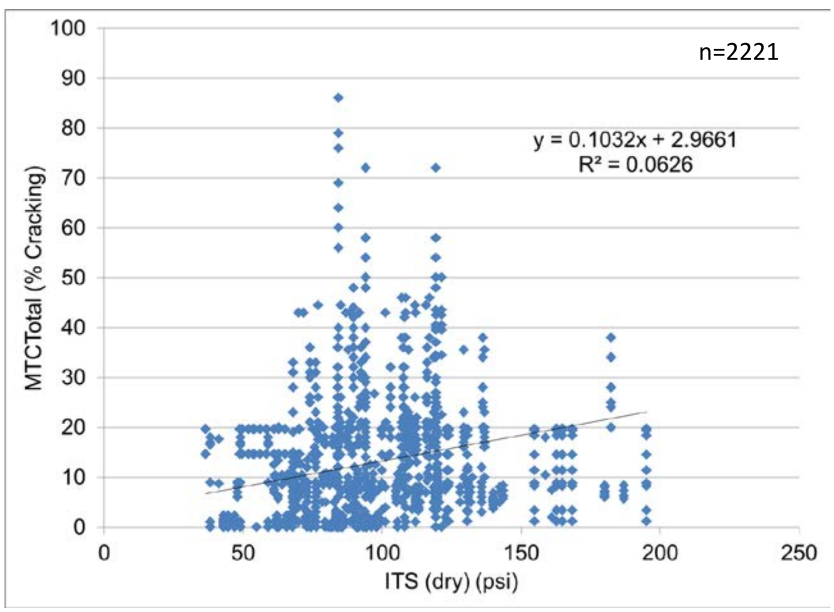

Figure 2.27: ITS (dry) and maximum total transverse cracking (MTCTotal)

2.3.3.1.2 ITS (wet)

The ITS (wet) data was analyzed in similar manner as the ITS (dry). The summary of statistical significance testing is presented in Table 2.24. The results for the ITS (wet) are analogous with those for ITS (dry). The transverse cracking measures (amount and rate) show statistically significant dependence on ITS (wet) and so do the longitudinal cracking amounts. However, the longitudinal cracking rates show minimal to no dependence on ITS (wet) of the asphalt mixes. The analysis of raw data (scatter) showed similar observations for ITS (wet) as ITS (dry), such that the data shows no consistent trend, have high amount of spread and a low coefficient of determination for the linear fit.

Table 2.24: Effect of ITS (wet) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to ITS (wet)?
MTCWeighted	0.0011	yes
MTCTotal	< 0.0001	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	0.0031	yes
MLCRTotal	0.6079	no
MLCRWeighted	0.0356	yes (weak)

2.3.3.1.3 TSR

The dependence of maximum field cracking measures on the TSR was determined using the linear regression analysis. The results from this analysis are tabulated in Table 2.25. As with ITS (dry and wet), the linear regression indicates that the cracking measures depend on TSR, except for the MTCWeighted. In order to further explore the dependence, a scatter plot has been generated between TSR and MTCTotal (Figure 2.28). Once again the plot unravels that while statistical test shows that MTCTotal is related to TSR, the change in MTCTotal with change in TSR is extremely small and the data has a large amount of spread. Similar observations were made for other transverse and longitudinal measures in context of TSR.

Table 2.25: Effect of TSR on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to TSR?
MTCWeighted	0.0647	no
MTCTotal	0.0070	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	0.0004	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	< 0.0001	yes

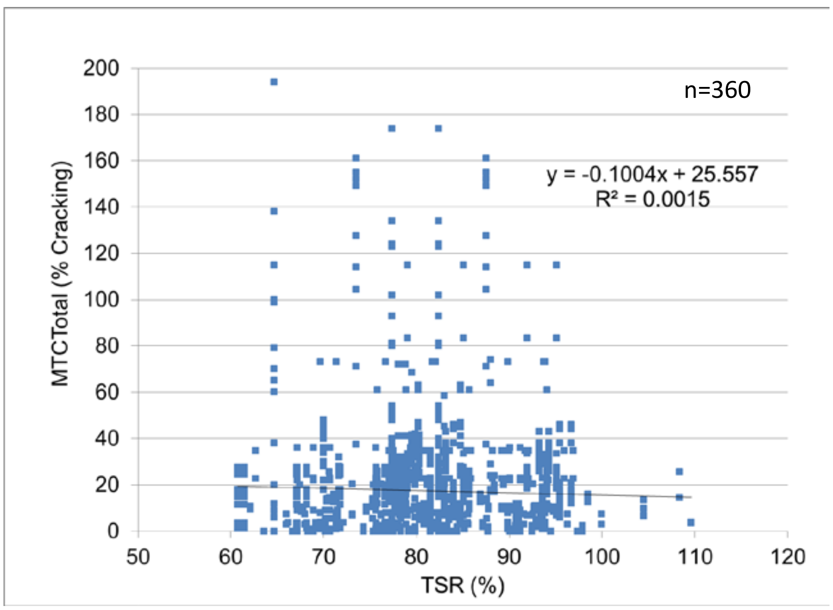

Figure 2.28: TSR and maximum total transverse cracking (MTCTotal)

2.3.3.2 Effects of ITS and TSR on Average Cracking Rates

The average longitudinal and transverse cracking rates were compared with the ITS (dry and wet) and TSR of asphalt mixes. The summary of the results are shown in Table 2.26. The results indicate that the ITS (dry and wet) have a statistically significant effect on the average transverse cracking rates whereas TSR does not. In order to quantify the extent of effect and to visualize the quality of fit between ITS and average transverse cracking rate, a scatter plot is presented in Figure 2.29. The scatter plot once again shows that while ITS (dry) is significant variable for ATCTotal, the amount of data scatter is very high and the fitted trend cannot be reliably used for purposes of prediction or as basis for development of specifications. The data analysis for ITS (wet) and TSR provided similar observations for average transverse cracking rates. The analysis of average longitudinal cracking rate data showed even greater spread in data, lower variation in cracking rate with changes in ITS (dry and wet) and TSR and lower coefficient of determination (R^2) for the linear fits.

Table 2.26: Effects of ITS (dry and wet) and TSR on measures of average field cracking rates

Mix Property	Average Cracking Rate	p-value	Field cracking rate is related to mix parameter?
ITS (dry)	ATC Total	< 0.0001	yes
	ATC Weighted	0.0027	yes
ITS (wet)	ATC Total	< 0.0001	yes
	ATC Weighted	0.0003	yes
TSR	ATC Total	0.0761	no
	ATC Weighted	0.3890	no
ITS (dry)	ALC Total	< 0.0001	yes
	ALC Weighted	0.0007	yes
ITS (wet)	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
TSR	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes

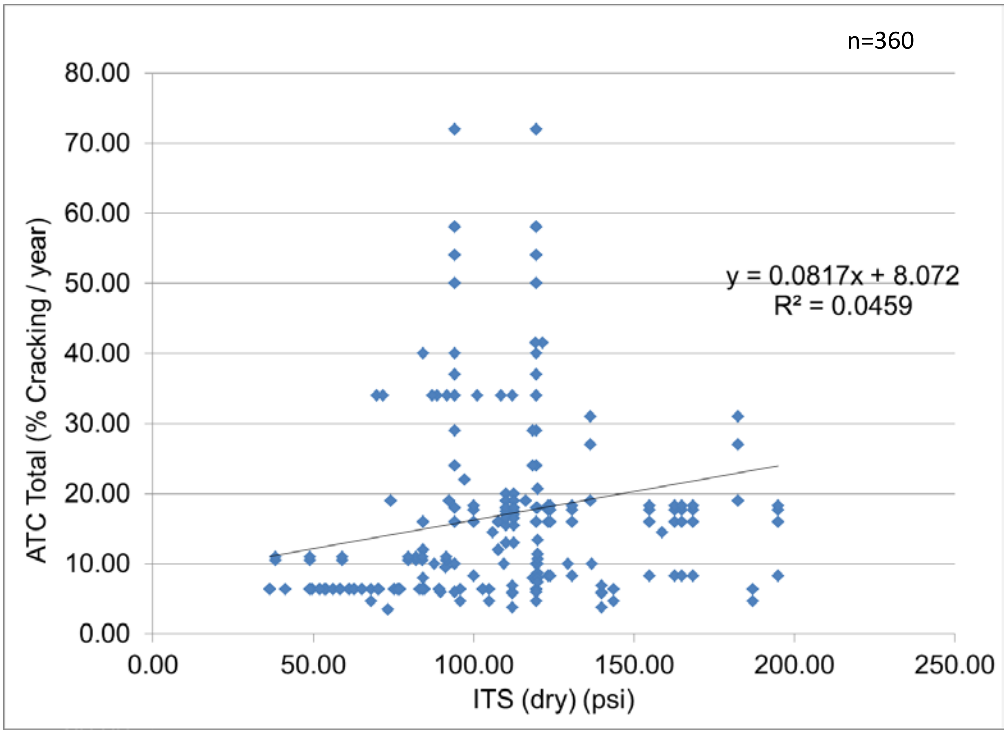

Figure 2.29: ITS (dry) and average total transverse cracking rate (ATCTotal)

2.3.4 Summary of Findings for use of ITS and TSR as Mix Performance Measures

Based on numerous data analysis described in this chapter of the report following key findings were inferred in context of using ITS (dry and wet) and TSR as asphalt mix performance measure:

- Most asphalt mix volumetric properties do not have significant effects on ITS and TSR.
- The asphalt binder grade has discernible effect on ITS, use of softer binder grade yields lower ITS values.
- The design traffic level (mix level) affects the ITS of the mix, with greater ITS values for higher traffic level mixes.
- Both ITS and TSR have a statistically significant effect on the asphalt pavement cracking performance. Mixes with higher ITS are expected to have a greater amount of cracking, and mixes with higher a TSR is expected to have lower amounts of cracking. The effect of ITS and TSR on the cracking amounts and cracking rates is relatively small.
- Both ITS and TSR are found to be poor candidates for use as performance measures for cracking in asphalt pavements. This is because of following reasons:
    - The ITS and TSR are independent of most asphalt mix design control measures (such as AFT or VMA), thus making them a difficult parameter to control for a mix designer.
    - The amount of scatter in the data is too high for ITS and TSR against the actual field cracking amounts and cracking rates. This will inherently lead to a very high

number of outliers that will not follow the trends predicted using statistical analysis.
- The variation in field cracking amounts with change in ITS and TSR was found to be relatively small to use either of these quantities as control measures.
- The pavement cracking performance improves with decrease in ITS of the mix. Thus, use of ITS as performance measure would require limiting the maximum value of ITS. Use of such limit would have detrimental effects on other asphalt mix durability and strength properties.

2.4 Effects of Mix Design Parameters on Field Cracking Performance

2.4.1 Introduction

This section describes the statistical analysis that was conducted to evaluate the effects of various asphalt mix design parameters on field cracking performance. The field cracking measures that were analyzed are same as those discussed previously in section 2.3. The mix design parameters that were studied herein include: Asphalt Film Thickness (AFT), Asphalt Binder Content (Percent Binder), Asphalt Binder Grade (PG Grade, PGLT and PG Spread), Presence of Recycled Materials and Voids in Mineral Aggregates (VMA). The field cracking performance measures that are used in this study include: Maximum Total Transverse Cracking Amount (MTCTotal), Maximum Total Weighted Transverse Cracking Amount (MTCWeighted), Maximum Total Transverse Cracking Rate (MTCRTotal), Maximum Total Weighted Transverse Cracking Rate (MTCRWeighted), Average Total Transverse Cracking Rate (ATCTotal), and Average Weighted Total Transverse Cracking Rate (ATCWeighted) for the transverse cracking. The corresponding field cracking measures for longitudinal cracking were also analyzed. The definitions of various cracking measures are provided in section 2.2 of this report.

The portion is divided into two main sections; the first section evaluates the effects of mix design parameters on maximum transverse and longitudinal cracking amounts and their rates (MTCTotal, MTCWeighted, MTCRTotal, MTCRWeighted, MLCTotal, MLCWeighted, MLCRTotal, MLCRWeighted). The second section compares the effect of mix parameters on average transverse and longitudinal cracking rates. While the data for these twelve field cracking measures were analyzed, the measure that is most relevant to this study is MTCWeighted, thus most graphical presentation of data is provided for this measure.

2.4.2 Analysis of BAB Pavements

The initial analysis of mix design parameter with respect to field cracking measures dealt with only conventional asphalt pavements (new construction or full reconstruction) which are typically referred to as Bituminous over Aggregate Base (BAB). The main reason to look only at BAB pavements was driven by the scope of this study which focused on transverse cracking in conventional asphalt pavements. Other pavement types included in the database are Bituminous on Bituminous (BOB), Bituminous on Stabilized Base (BSD), and Bituminous on Concrete (BOC).

Upon generating queries within the database to only return BAB pavements, it was found that an insufficient amount of records were returned for several mix parameters. This can be seen below in Table 2.27. For adjusted AFT, unadjusted AFT (AFT P_{be}), asphalt binder content, presence of recycled materials, and VMA, there were not enough results to conduct a reliable statistical analysis. Due to lack of sufficient amount of data for exclusively BAB pavements, the analysis to evaluate the effects of mix design parameters on the pavement cracking performance was conducted using all pavement types that have a bituminous surface course (BAB, BOB, BOC, and BSD). The subsequent sections present the effectiveness of mix design parameters on maximum and average cracking amounts and rates for all pavements with a bituminous surface.

Table 2.27: Average transverse cracking statistical analysis on BAB pavements

Mix Parameter	Average Cracking Rate	p-value	Field cracking rate is related to mix parameter?
Adjusted AFT	ALC Total	-	Not enough data
	ALC Weighted	-	Not enough data
AFT P_{be}	ALC Total	-	Not enough data
	ALC Weighted	-	Not enough data
Percent Binder (Extracted)	ALC Total	-	Not enough data
	ALC Weighted	-	Not enough data
Percent Binder (Ignition)	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
PG Grade	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
PGLT	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
PG Spread	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
Presence of Recycled Material	ALC Total	-	Not enough data
	ALC Weighted	-	Not enough data
VMA (Extracted)	ALC Total	-	Not enough data
	ALC Weighted	-	Not enough data
VMA (Ignition)	ALC Total	0.0002	yes
	ALC Weighted	< 0.0001	yes

2.4.3 Effect of Mix Parameters on Maximum Cracking Amounts and Rates

2.4.3.1 Asphalt Film Thickness (AFT)

The effect of AFT on the field cracking measures were evaluated for both: adjusted AFT and AFT calculated based on the effective binder content (P_{be}). The results from the statistical analysis on the data are presented in Table 2.28 and Table 2.29. It can be seen that the measures for both transverse and longitudinal cracking show dependence on adjusted AFT, whereas only some of the longitudinal cracking measures show dependence on unadjusted AFT calculated from P_{be}.

Table 2.28: Effect of adjusted AFT on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to adjusted AFT?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	< 0.0001	yes

Table 2.29: Effect of AFT (P_{be}) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to AFT (P_{be})?
MTCWeighted	0.323	no
MTCTotal	0.751	no
MTCRTotal	0.5787	no
MTCRWeighted	0.3101	no
MLCTotal	0.037	yes
MLCWeighted	0.0708	no
MLCRTotal	< 0.0001	yes
MLCRWeighted	0.0006	yes

To further investigate the effect of adjusted AFT on the extent of transverse cracking, the data was converted to a normalized frequency. This was accomplished by first distributing the adjusted AFT into discrete intervals of 0.5 micron increments. These increments were assigned a character key, as seen in Figure 2.25. Next, the data for all pavement sections in each range of adjusted AFT data were analyzed to determine the percent of sections that have no cracking

(MTCWeighted of 0 %/500 ft./year) and ones with cracking at 10, 20, 30, 40 and 50 %/500 ft./year respectively. The database consists of very few pavements with cracking amounts above 50 %, therefore cracking amounts above this percentage were not considered. Thereafter, the normalized frequencies for each of the cracking amounts were plotted for each adjusted AFT range. Figure 2.30 shows the normalized frequencies for MTCWeighted for various adjusted AFT levels.

It can be seen from the plot that for pavements without any transverse cracking (MTCWeighted = 0 %/500 ft./year), as the adjusted AFT increases, the percentage of crack-free pavements decreases. For example, the data shows that for mixes with adjusted AFT between 7.0 and 7.5 micron, approximately 12% pavements are crack free, whereas for mixes with adjusted AFT between 8.0 and 8.5 micron only 5% pavements are free of transverse cracking. For the pavements where cracks are present (10, 20, 30, 40 and 50 %/500 ft./year), no specific trends are observable with the level of adjusted AFT. Thus in summary, while there is a specific trend between adjusted AFT and transverse cracking performance, it is only limited to pavements without any cracks. Once the cracks start to develop the adjusted AFT does not correlate with the cracking performance.

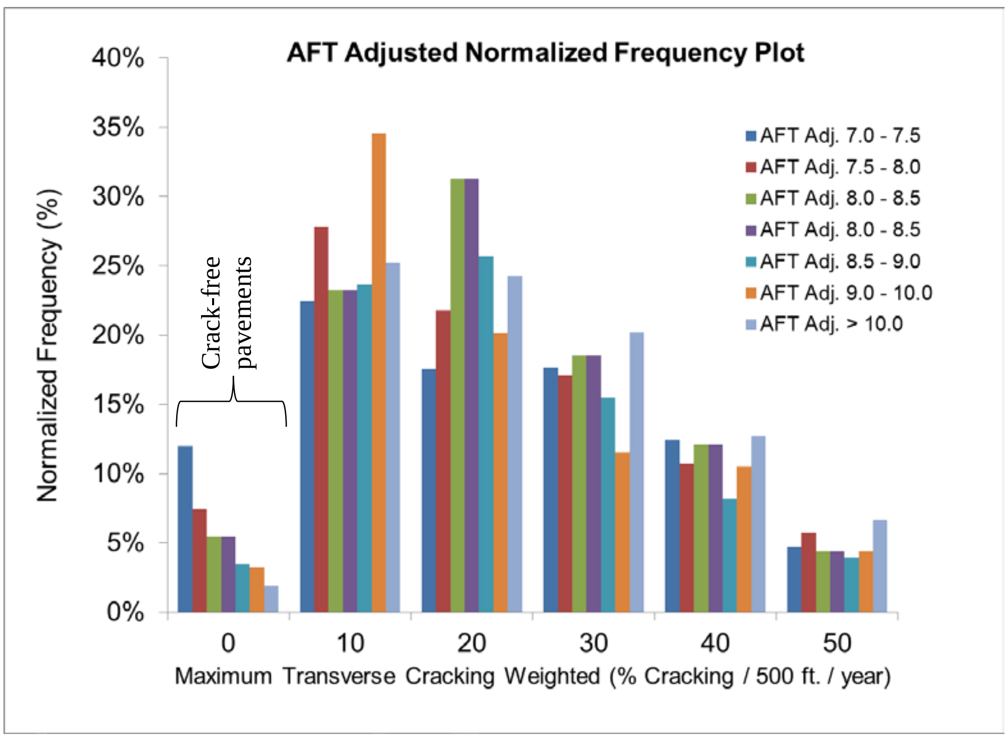

Figure 2.30: Normalized frequency plot of adjusted AFT with various ranges of weighted maximum transverse cracking amounts (MTCWeighted)

2.4.3.2 Asphalt Binder Amount (Percent Binder)

The asphalt binder contents determined using chemical extraction and ignition oven methods were used to determine their effects on the measures of transverse and longitudinal cracking. The statistical analysis for asphalt binder contents determined using both methods are presented in Table 2.30 and Table 2.31. The results show that irrespective of the measurement method, the asphalt binder content has significant effect on the amount of field cracking (both transverse and longitudinal).

In order to further evaluate this effect, the MTCWeighted data was analyzed to determine the normalized frequencies. The normalized frequencies of the transverse cracking amounts for various ranges of asphalt binder contents are plotted in Figure 2.31 for chemical extraction and Figure 2.32 for ignition oven methods. The results show that in general, a greater percent of pavements are free of transverse cracks for mixes with higher asphalt binder content. The data for mixes with greater than 6.0% asphalt binder content as determined using chemical extraction is the only outlier. The plots also show that the number of pavements with 20, 30 and 40 %/500 ft./year cracking increase as the amount of asphalt binder in mixes decrease. Specifically, the asphalt mixes with binder contents between 4.0 and 4.5% represent almost 10% more pavements with 20 and 30 %/500 ft./year cracking as compared to other mixes.

Table 2.30: Effect of asphalt binder content (chemical extraction) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to percent binder?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	0.0005	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	< 0.0001	yes

Table 2.31: Effect of asphalt binder content (ignition oven) on measures of maximum field cracking

racking Measure	p-value	Field cracking is related to percent binder?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	< 0.0001	yes

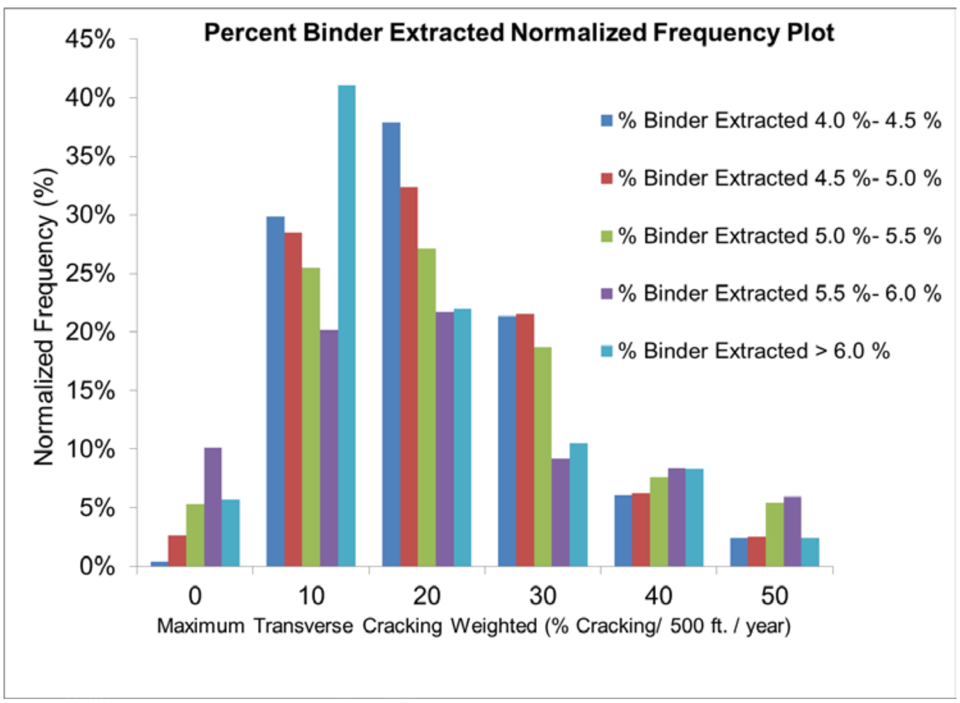

Figure 2.31: Normalized frequency plot of asphalt binder content (chemical extraction) with various ranges of weighted maximum transverse cracking amounts (MTCWeighted)

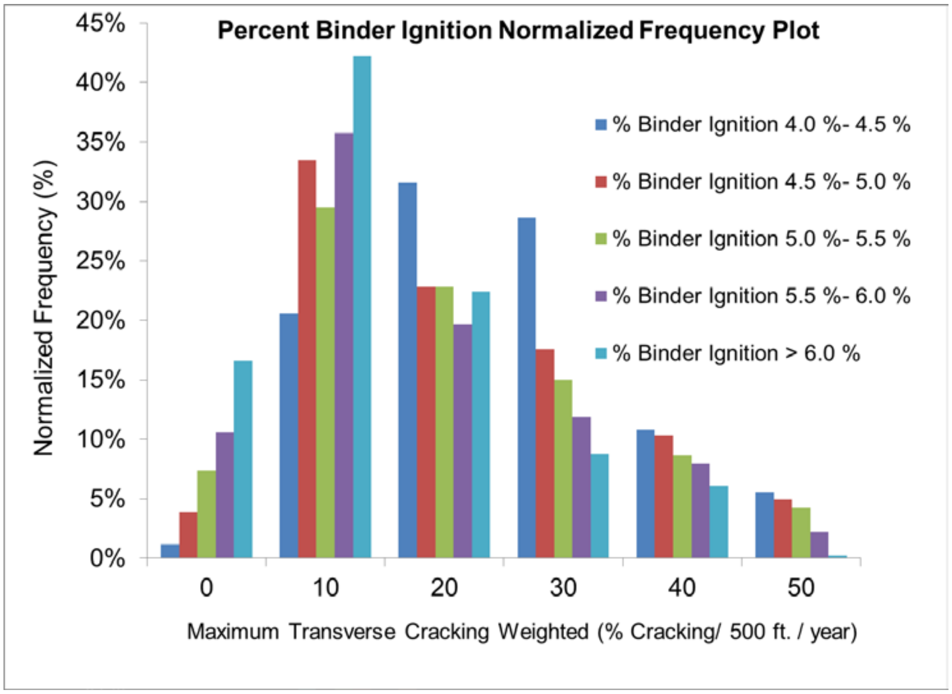

Figure 2.32: Normalized frequency plot of asphalt binder content (ignition oven) with various ranges of weighted maximum transverse cracking amounts (MTCWeighted)

2.4.3.3 Asphalt Binder Grade (PG, PGLT and PG Spread)

The asphalt binder grade (PG), the low temperature grading of the binder (PGLT) and the spread between the high and low temperature grading (PG Spread) information was used to conduct a statistical analysis. The results from the statistical analysis conducted to determine whether different field cracking measures depend on the asphalt binder grade and its derivatives are presented in Table 2.32, Table 2.33 and Table 2.34. The results show that the asphalt binder grade has significant effect on the amounts and rates of transverse and longitudinal cracking. The only exception is non-dependence of total amount of longitudinal cracking on the low temperature grade of the binder (PGLT).

Table 2.32: Effect of asphalt binder grade (PG) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to PG?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	< 0.0001	yes

Table 2.33: Effect of asphalt binder low temperature grade (PGLT) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to PGLT?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	0.0602	no
MLCWeighted	0.0035	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	< 0.0001	yes

Table 2.34: Effect of spread in asphalt binder grade (PG Spread) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to PG Spread?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	< 0.0001	yes

Similar to analysis of previous mix parameters, in order to determine the effects of asphalt binder grade on the amount of cracking, the PGLT data was used to generate normalized frequencies of the maximum weighted transverse cracking (MTCWeighted). The frequencies are plotted for PGLT of -28 and -34 °C. These two were selected as a majority of pavements in the database represent these types of binders. The results are plotted in Figure 2.33. The results show that significantly greater amount of pavements are crack free when containing mix with PGLT of -34. Furthermore larger number of pavements with higher transverse cracking amounts (20, 30 and 40 %/500 ft./year) have mixes with PGLT -28 binder.

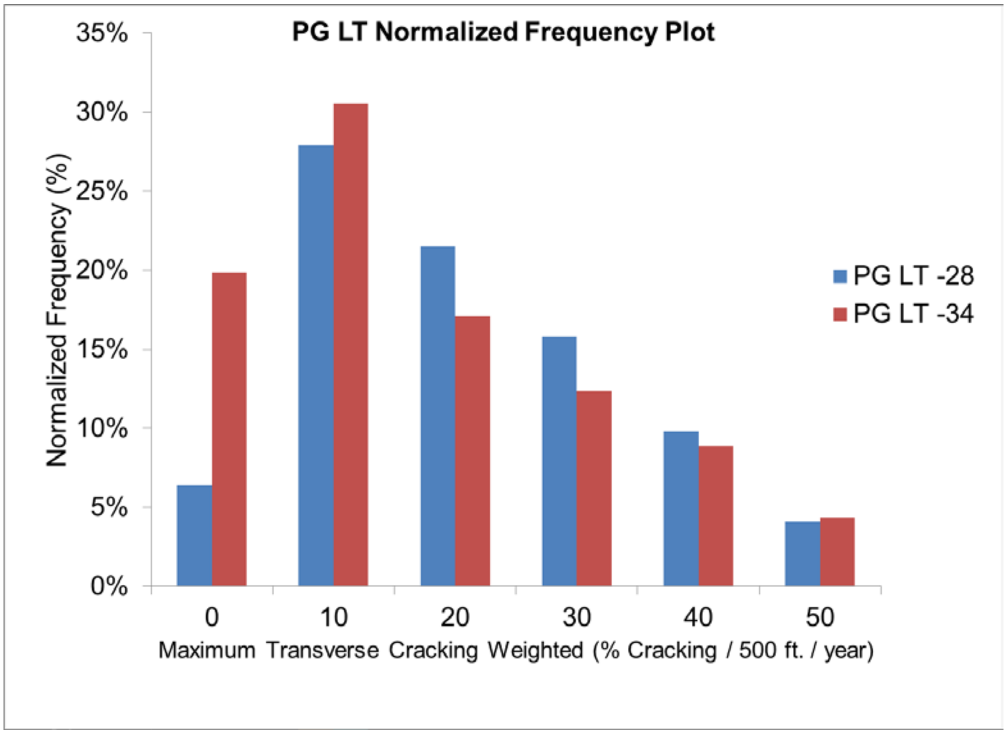

Figure 2.33: Normalized frequency plot of asphalt binder low temperature grade (PGLT) with various ranges of weighted maximum transverse cracking amounts (MTCWeighted)

2.4.3.4 Presence of Recycled Materials

The asphalt mixes in the database that consisted of either reclaimed asphalt pavement (RAP) or recycled asphalt shingles (RAS) or both were differentiated from the mixes without inclusion of these products. Thereafter, the field cracking performance for all mixes in each category was determined. In total 432 pavement sections were identified with mixes containing no recycled materials versus 27877 sections with recycled materials. This indicates the widespread use of recycled materials in the asphalt mixes. The statistical analysis was conducted to determine whether presence of recycled material in asphalt mixes had a discernible effect on the field cracking performance. The results from statistical analysis are tabulated in Table 2.35. The results show that the maximum transverse and longitudinal cracking amounts (MTCTotal, MTCWeighted, MLCTotal and MLCWeighted) are related to the presence of recycled materials. The cracking rates are not related to presence of recycled materials except in case of maximum weighted transverse cracking rate (MTCRWeighted).

It should be noted that it was not possible to screen out the amount of recycled materials as the recycled material stockpile number in LIMS is variable and can change between mixes. The process to find out the amount of recycled materials in the mixes would require manual screening of each mix record. The associated time requirement with this was prohibitive.

The normalized frequencies were generated for various ranges of MTCWeighted for mixes with and without recycled materials. The normalized frequencies are plotted in Figure 2.34. It should be noted that the amount of data for pavements without recycled materials is quite small as compared to those with recycled materials (432 versus 27877), thus these results should be treated as preliminary. The results show that a large percent of pavements with all virgin mixes are crack-free as compared to pavements with mixes containing recycled materials (32% for virgin mixes versus 10% for mixes with recycled materials). This trend is not consistent for the pavements with transverse cracking.

Table 2.35: Effect of presence of recycled material on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to presence of recycled material?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	0.2549	no
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	0.2784	no
MLCRWeighted	0.1367	no

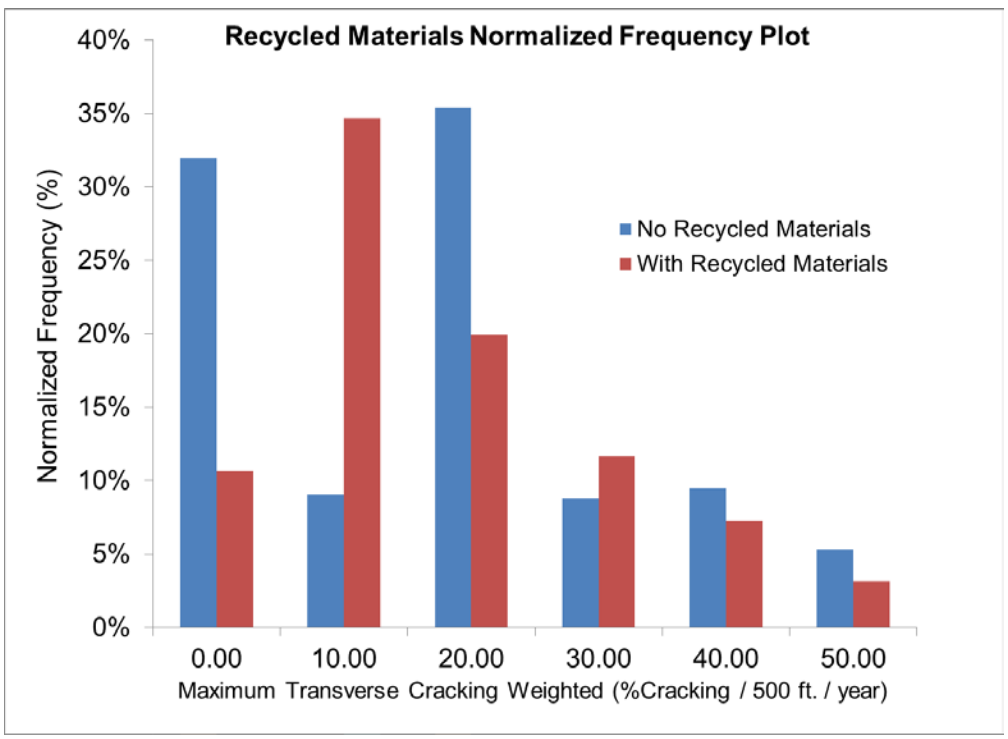

Figure 2.34: Normalized frequency plot of presence of recycled materials with various ranges of weighted maximum transverse cracking amounts (MTCWeighted)

2.4.3.5 Voids in Mineral Aggregates (VMA)

The VMA amounts determined using the chemical extraction and ignition oven methods were compared with the field cracking measures to determine if VMA showed a statistically significant relationship with cracking performance. The results from statistical testing are tabulated for VMA determined using chemical extraction and ignition oven methods in Table 2.36 and Table 2.37 respectively. The field cracking measures show statistically significant relationship with both types of VMA. Similar to percent asphalt binder content, the significance is weaker for certain field cracking measures when the VMA calculation was based on chemical extraction. This is not surprising since the chemical extraction method provides the asphalt binder content, which is in-turn used to calculate VMA. The normalized frequency plots for the VMA are presented in Figure 2.35 and Figure 2.36. The normalized frequencies show that mixes with low VMA correspond to fewer pavements that are free of transverse cracks. The trends are not completely consistent for pavements containing cracks to make a general statement regarding preference towards low or high VMA.

Table 2.36: Effect of VMA (chemical extraction method) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to VMA (extracted)?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	0.0456	yes
MTCRWeighted	0.0006	yes
MLCTotal	< 0.0001	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	0.0005	yes

Table 2.37: Effect of VMA (ignition oven method) on measures of maximum field cracking

Cracking Measure	p-value	Field cracking is related to PG Spread?
MTCWeighted	< 0.0001	yes
MTCTotal	< 0.0001	yes
MTCRTotal	< 0.0001	yes
MTCRWeighted	< 0.0001	yes
MLCTotal	< 0.0001	yes
MLCWeighted	< 0.0001	yes
MLCRTotal	< 0.0001	yes
MLCRWeighted	< 0.0001	yes

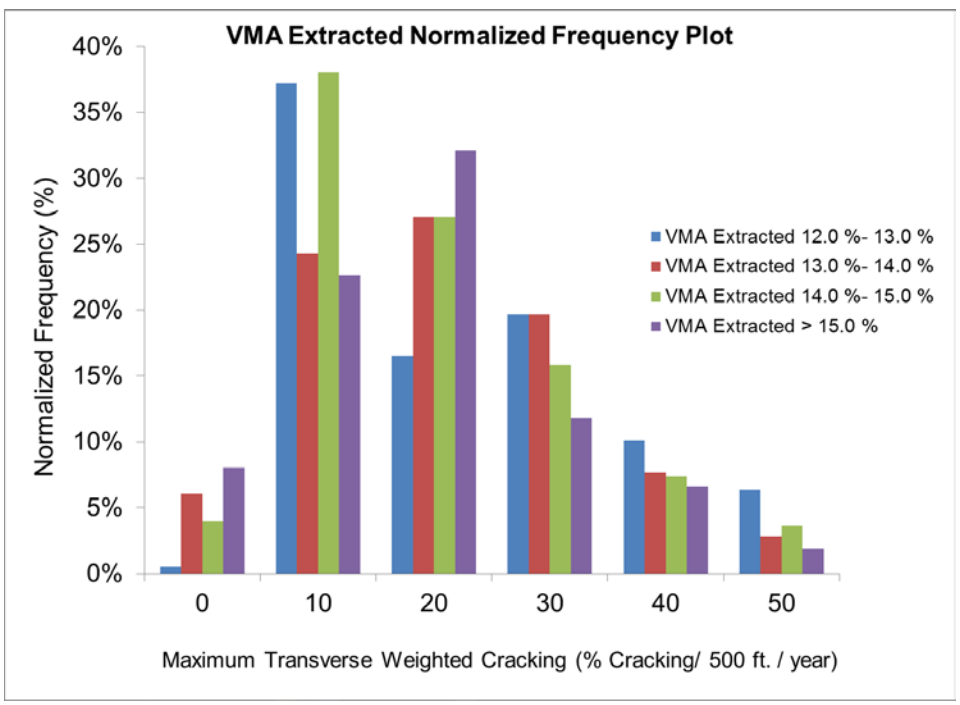

Figure 2.35: Normalized frequency plot of VMA (chemical extraction) with various ranges of weighted maximum transverse cracking amounts (MTCWeighted)

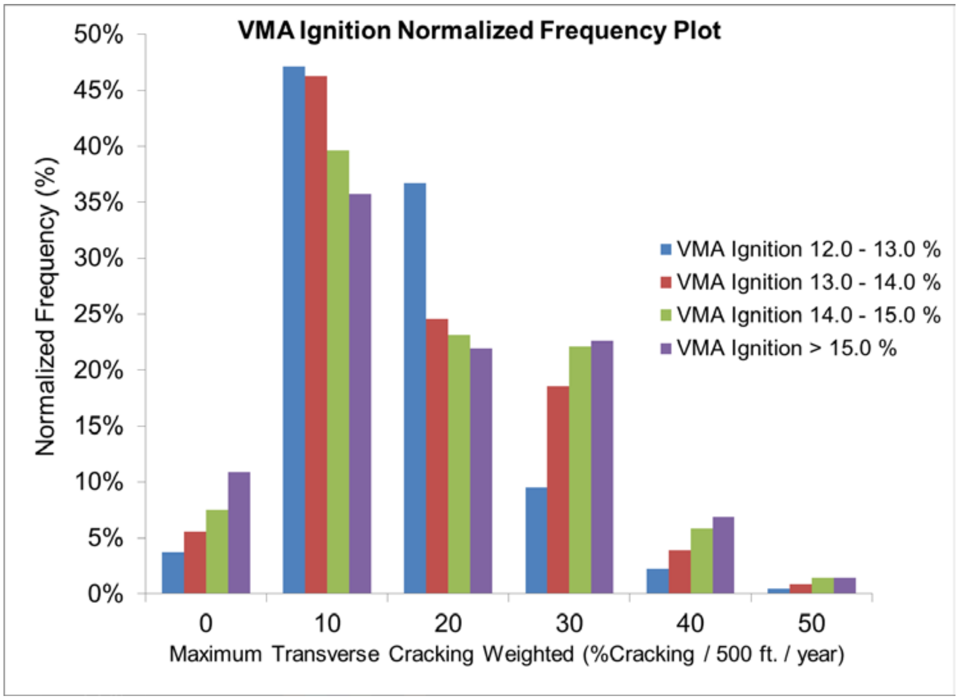

Figure 2.36: Normalized frequency plot of VMA (ignition oven) with various ranges of weighted maximum transverse cracking amounts (MTCWeighted)

2.4.4 Effects of Mix Design Parameters on Average Cracking Rates

The average transverse and longitudinal cracking rates were statistically tested against various mix design parameters that were discussed in the previous section. The main reason for this set of analysis was to determine whether the findings from previous section, which dealt with looking at the maximum cracking amounts and maximum cracking rates, were applicable when looking at rate of crack development as average over the survey period for a pavement section.

The statistical testing was conducted to determine the significance of various mix design parameters in affecting the average cracking rates. The results are presented for the average transverse cracking rates (both total: ATC Total and weighted: ATC Weighted) in Table 2.38. Similar to the maximum cracking amount and rate, the average transverse cracking rates show a statistically significant relationship to almost all mix design parameters except the unadjusted AFT as calculated from P_{be}. The overall trends of data for the average transverse cracking rate were similar to the maximum transverse cracking amounts and rates for various mix design parameters.

Table 2.38: Effect of mix design parameters on average transverse cracking rates

Mix Parameter	Average Cracking Rate	p-value	Field cracking rate is related to mix parameter?
Adjusted AFT	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes
AFT P_{be}	ATC Total	0.4948	no
	ATC Weighted	0.4835	no
Percent Binder (Extracted)	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes
Percent Binder (Ignition)	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes
PG Grade	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes
PGLT	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes
PG Spread	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes
Presence of Recycled Material	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes
VMA (Extracted)	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes
VMA (Ignition)	ATC Total	< 0.0001	yes
	ATC Weighted	< 0.0001	yes

The average longitudinal cracking rates for the pavement sections were statistically analyzed to determine if they were significantly related to various mix design parameters. The results from this set of analysis are tabulated in Table 2.39. The results show that the average longitudinal cracking rates are related to all mix design parameters except the PGLT. The dependence of average longitudinal cracking rates on unadjusted AFT is not as strong as other mix design parameters. Unlike the maximum longitudinal cracking rate, the average rate depends on the presence of recycled materials.

Table 2.39: Effect of mix design parameters on average longitudinal cracking rates

Mix Parameter	Average Cracking Rate	p-value	Field cracking rate is related to mix parameter?
Adjusted AFT	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
AFT P_{be}	ALC Total	0.0031	yes
	ALC Weighted	0.0112	yes
Percent Binder (Extracted)	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
Percent Binder (Ignition)	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
PG Grade	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
PGLT	ALC Total	0.7698	no
	ALC Weighted	0.1804	no
PG Spread	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
Presence of Recycled Material	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
VMA (Extracted)	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes
VMA (Ignition)	ALC Total	< 0.0001	yes
	ALC Weighted	< 0.0001	yes

2.4.5 Summary of Effects of Mix Design Parameters on Field Cracking Performance

From the statistical analysis as well as the normalized frequency analysis, the following findings were realized to describe the effects mix design parameters on field cracking performance:

- Both transverse and longitudinal cracking depend on adjusted AFT, while only some longitudinal cracking measures depend on the unadjusted AFT
    - AFT adjusted with lower values correlate to a higher amount of pavements with no transverse cracking (MTCWeighted = 0 %/500 ft./year), There was no

observable trend with pavements containing cracking amounts of 10, 20, 30, 40, or 50 (%/500 ft. / year).

- The amount of asphalt binder, both calculated by means of chemical extraction and ignition oven, showed significant effect on both transverse and longitudinal field cracking performance
    - o Mixes with higher asphalt contents corresponded to a greater percent of pavements with low transverse cracking. Specifically with pavements containing 20, 30, or 40 (%/ 500 ft./year), the cracking amounts are higher for with mixes containing low amounts of asphalt binder.
- The PG binder grade, PGLT, and PG Spread do have an effect on transverse and longitudinal cracking amounts, with the exception of PGLT which showed minimal effect on the amount of longitudinal cracking
    - o Mixes with PGLT -34 have higher amounts of transverse crack free pavements as compared the mixes with PGLT -28. A higher amount of 20, 30, and 40 (%/ 500 ft./ year) cracking occurred for mixes with PGLT -28 binder.
- Only a small amount of pavements with no recycled materials were available for analysis and these results should be treated as preliminary findings
    - o Maximum transverse and longitudinal cracking (MTCTotal, MTCWeighted, MLCTotal, and MLCWeighted) showed dependence on presence of recycled materials;
    - o A large percentage of pavements containing no recycled materials were crack free (MTCWeighted 0%/500ft./year); and,
    - o The trend of virgin mixes having low amounts of cracking was not consistent within the MTCWeighted normalized frequency plot.
- VMA (determined by both chemical extraction and ignition oven) showed a statistically significant relationship with transverse and longitudinal field cracking
    - o Pavements with lower VMA values resulted in fewer pavements with no transverse cracking. This trend was not consistent with cracking of 20, 30, 40, and 50 (%/ 500ft. / year).
- Average transverse cracking showed relationship to all mix design parameters except for unadjusted AFT
    - o Trends for average transverse cracking were similar to both maximum transverse cracking amounts and maximum transverse cracking rates (MTCTotal/Weighted, MLCTotal/Weighted, MTCRTotal/Weighted, MLCRTotal/Weighted) for various mix design parameters.
- Average longitudinal cracking is related to all mix design parameters except PGLT
    - o Average longitudinal cracking does depend on the presence of recycled materials, unlike MLCRTotal and MLCRWeighted.

2.5 Summary, Conclusions and Recommendations

2.5.1 Summary

The Task-1 (Analysis of Laboratory Test and Field Performance Data) of the MnDOT research contract 99008, work order 40 was completed. The task undertook three primary research efforts:

(1) Development of a comprehensive database that includes asphalt material property data (mix design records), bituminous pavement construction information (SP information, location, construction year) and the pavement management information (section locations, survey years, cracking data). The development of the comprehensive database is described in Chapter 2 of this report. The database was developed using the Microsoft Access software. The database is delivered in the electronic format along with this report.

(2) The second effort involved determination of whether the indirect tensile strength (ITS) from the modified Lottman test (AASHTO T-283) can be used as a cracking performance measure. The modified Lottman test is conducted routinely as part of the mix design process, thus if ITS can be used as a performance measure a new specification control could be added with minimal additional testing requirements. The evaluation of ITS as performance measure was done in two phases. The first phase evaluated whether ITS is dependent on asphalt mix design parameters such as, binder grade, binder content, volumetric measures, mix size and design traffic level. The second phase evaluated whether ITS of mix has a statistically significant effect on the field cracking performance as well as the consistency with which ITS affects the field cracking performance.

(3) The effects of mix design parameters (mix volumetrics, mix design (traffic) level, asphalt binder amounts and grades, use of recycled materials) on the pavement cracking performance was evaluated. The findings from this effort allow identification of mix design parameters that affect pavement cracking performance. The study also determined the effects of the mix design choices on the cracking performance such as, use of -28 grade asphalt binder as compared to -34 grade binder.

The analyses conducted in above listed efforts 2 and 3 always dealt with very large amounts of data (over 12,000 material records and over 58,000 pavement management data points were included). For such large data it is necessary to use statistical analysis for determining effects of one parameter on other. It is also important to note that the statistical testing for significance between two parameters should be scrutinized before drawing conclusions. In this study, the parameters that showed a statistically significant relationship were further evaluated to determine the strength of relationship and the extent to which one parameter affected the other.

Based on the three efforts listed above, a number of findings were made. These are described in detail at the end of sections 2.3 and 2.4. The key conclusions drawn from this study and the corresponding recommendations are discussed in the subsequent sections.

2.5.2 Conclusions

The findings from this study resulted in several conclusions regarding the ITS of asphalt mixes and the effects of mix design parameters on field cracking performance of asphalt pavements. Please note that these conclusions are limited for traditional hot mix asphalt manufactured according to MnDOT 2360 specifications. Also notice that a small number of traditional asphalt pavements (BAB: bituminous on asphalt base) are constructed in past decade, thus the analyses conducted in this study included pavements with all types of asphalt surfaces (BAB, BOB, BOC and BSD). The key conclusions drawn from this study are as follows:

- The indirect tensile strength (ITS) of the asphalt mixes, as determined using the AASHTO T-283 specifications, is found to be a poor measure of pavement cracking performance.
- A higher percentage of crack free pavements were represented by asphalt mixes that have lower adjusted asphalt film thickness (AFT) and higher voids in mineral aggregates (VMA). For pavements that have cracks present in them, neither adjusted AFT nor VMA showed consistent trends.
- Asphalt binder grade has a significant effect on the pavement cracking performance. Mixes containing -34 asphalt binders have significantly greater amount of crack-free pavements as compared to mixes containing -28 binders. Fewer percent of pavements with significant amounts of transverse cracking are represented by mixes with -34 binder grades as compared to those with -28 binder grades.
- The amount of asphalt binder has a significant effect on field cracking performance. The mixes with higher asphalt content showed lower amounts of cracking.
- Very few pavements constructed with all virgin materials were present in the database, thus limited data was available to make final conclusions regarding presence of recycled materials on cracking performance. For the limited data, a larger fraction of crack free pavements are represented by all virgin mixes as compared to mixes containing recycled materials.
-

2.5.3 Recommendations

The generation of a comprehensive database and the subsequent statistical analysis of ITS and asphalt mix design parameters in context of field cracking performance helped make several observations regarding future recommendations. The key recommendations from the research efforts of this study are as follows:

- The development of a comprehensive database required developing an extensive search algorithm to map the cracking data from pavement management highway sections onto the material records from the laboratory information system and the construction records. If the future versions of the pavement management system can include a variable that tracks the highway construction information (for example, project SP), the development of a comprehensive database, such as one developed in this study, will require significantly fewer amount of human resources and computational efforts.
- The asphalt binder amount and grade play an important role in the cracking performance of bituminous pavements and overlays. The asphalt binder grade recommendations along with the potential for use of a minimum asphalt binder amount in the specifications should be reevaluated. The future tasks of the current project will provide additional information on this topic through field and laboratory evaluation of several pavement sections.

- It was not possible to analyze the effects of the amount of recycled materials on pavement cracking performance. A major challenge is the quantification of the percent of recycled material in asphalt mix which will require manual scrutiny of each mix design record, one at a time. If the future version of mix design records can be modified to explicitly report the percent of recycled materials, the future data analysis can be automated to analyze the effects of amount of recycled materials on pavement performance.
- In general, the volumetric quantities determined using chemical extraction process (binder content, VMA, VFA etc.) showed inferior correlation with ITS and field cracking amounts as compared to same quantities determined using the ignition oven method. This is very peculiar, as the volumetric quantities as anticipated to be comparable between the two binder content determination methods. The data should be further analyzed to determine if there exists a consistent bias between two methods.
- The data analysis presented herein did not normalize the field cracking performance measured against the amount of traffic. The future data analysis should consider this effect to determine if the cracking amounts and rates are significantly affected by traffic level and whether the effects of mix design parameters on cracking are altered by the effects of traffic.

CHAPTER 3: CRACKING PERFORMANCE EVALUATION OF FIELD SECTIONS (TASK-2A)

3.1 Overview of Task-2A

The Task-2A of the "Laboratory Performance Test for Asphalt Concrete" project involved field evaluation of several roadways across Minnesota. Nine asphalt roadways were chosen for this study through interactions with the technical advisory panel (TAP) for the project. The projects were chosen to obtain a wide cross-section of varying asphalt mixture designs and pavement structures. During the course of this task construction plans were evaluated, site visits were conducted, field cracking performance was determined and field sampling plans were developed. This report will provide an overview of the individual site visits, cracking performance information for each pavement section and the field sampling plans.

Chapter 4 of this report provides the laboratory testing results or the field cores. The discussions pertaining to comparisons between the mix design parameters to field cracking amounts are discussed in Chapter 5 of the report. It is recommended that readers review this chapter along with Chapter 4 and 5 to get the comprehensive information.

3.2 Field Performance Evaluation

3.2.1 Field Sites

In order to study the effects of the asphalt mix parameters on the field cracking performance, as well as to assess the suitability of laboratory performance tests in predicting cracking performance, a total of nine highway projects were selected. The field sites were selected through the interactions between the researchers and the technical advisory panel for the project. The designs for the nine sites varied greatly; traffic level, climatic conditions and wear course thickness differed between each of the sites. The construction plans for each of the pavement sections are attached in the appendix to this report. The location of the sections with respect to MnDOT district layout is shown in Figure 3.1. The sections are located along the following highways:

- Trunk Highway 1 (District 1)
- Trunk Highway 2 (District 2)
- Trunk Highway 6 (District 2)
- Trunk Highway 10 (District 3)
- Interstate 35 (Metro)
- Trunk Highway 53 (District 1)
- Trunk Highway 113 (District 2)
- Trunk Highway 210 (District 3)
- Trunk Highway 212 (Metro)

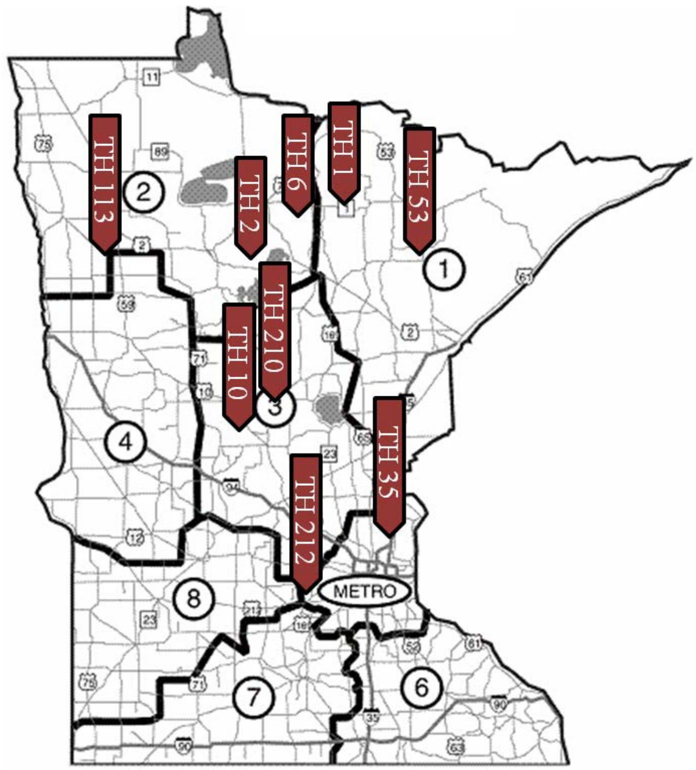

Figure 3.1: Locations of Test Sections

3.2.2 Field Evaluation Procedures

The aim of field visits was twin fold: (1) conduct crack surveys to quantify the cracking performance; and, (2) develop sampling plans for sample procurement. The information available through the MnDOT pavement management system identifies transverse cracking severity along with percent cracking determined through number of cracks in a pavement section. Researchers in this study conducted the manual cracking surveys and hence were able to measure the actual length of cracking in each study section, however the severity of cracks in terms of crack width was not documented. Chapter four of this report explains various cracking measures that were used for data analysis.

When first arriving at a site, the researchers conduct a full length evaluation of the desired section by driving the entire length of a project (Project is defined as the stretch of highway constructed under same SP number). In most situations, a good performing and poor performing portion of the roadway are identified. These areas are treated as distinct study sections. Also, in instances where two different mixes or pavement structures are used (such as, mill and overlay versus reclaim), distinct sections are used. These areas are noted and returned to at the conclusion of the full length review. A reference post (or mile post), herein referred to as RP, or landmark is found in the area(s) of interest and used as a starting point for field evaluation. The evaluation consists of inspecting a 1000 foot section, measured from the desired RP. Along this 1000 foot section, any transverse cracks are measured in both length of crack and distance from the starting point. The data sheets used for these field evaluations can be found in the appendix.

Field cores must be taken at these sites for disk-shaped compact tension (DCT) testing. The locations for coring are recommended to be at 200 foot increments along the 1000 foot inspection corridor. During the field visits the GPS coordinates are recorded at the start, end and each coring location in the survey section. This results in five coring locations per inspection site. The total number of cores that will be extracted varies relative to wear course depth. Coring plans are developed for each study section, these plans can be found in the appendix of this report.

3.2.3 Field Visit Summary

The site visit dates for various projects are indicated below. Due to very high traffic levels and consequently inability to obtain traffic control, a formal site visit was not conducted at I-35 and Trunk Highway 212. The pavement management data from these pavements is available and used in the analysis. The cored samples will also be extracted from the roadways and included in the study.

- Trunk Highway 1 (District 1): June 18, 2014
- Trunk Highway 2 (District 2): January 3, 2014
- Trunk Highway 6 (District 2): January 3, 2014
- Trunk Highway 10 (District 3): October 17, 2013
- Interstate 35 (Metro): December 12, 2012 (drive through survey)
- Trunk Highway 53 (District 1): June 18, 2014
- Trunk Highway 113 (District 2): January 2, 2014
- Trunk Highway 210 (District 3): January 8, 2014
- Trunk Highway 212 (Metro): No site visit conducted

Table 3.1 summarizes the information from the site visits along with pertinent information from the construction plans. Please note that the performance measures of "GOOD" and "POOR" are qualitative and were used to setup distinctly different study sections. The actual performance was determined using crack counts and pavement management data and is discussed later in this report and used in the analysis conducted through Task-3A of the project.

Any cells in Table 3.1 listed as "N/A" are classified as such for one of the following reasons:

- RP/Landmark: no formal site visit was conducted at this site due to high traffic conditions
- Performance: there was no substantial difference throughout the entire section and one survey was sufficient; or no formal site visit was conducted at this site due to high traffic conditions
- Lane: no formal site visit was conducted at this site due to high traffic and no historical data was immediately available

Table 3.1: Summary of Site Visits

Section	SP #	RP / Landmark	Construction Year	Performance	Lane	Construction Type
TH 1	8821-103	RP 235	2008	Poor	D	1.5" O/L on old AC
TH 1	8821-103	RP 230	2008	Good	D	4" O/L on reclaimed AC
TH 2	1102-59	RP 157	2003	N/A	D	4" O/Lon old AC
TH 6	3107-42	RP 118	2004	Poor	D	1.5" O/L on old AC
TH 6	3107-42	RP 123	2004	Good	D	4.5" O/L on reclaimed AC
TH 10	0502-95	RP 159	2005	Poor	D	4" M/O (sealed cracks)
TH 10	0502-95	RP 159	2005	Poor	P	4" M/O (sealed cracks)
TH 10	0502-95	RP 161	2005	Good	D	4" M/O (cracks not sealed)
TH 10	0502-95	RP 161	2005	Good	P	4" M/O (cracks not sealed)
*I-35	0283-26	N/A	2009	N/A	N/A	4" M/O on existing concrete
TH 53	8821-177	169 to Ely	2008	N/A	D	1.5" M/O
TH 53	8821-177	169 to Ely	2008	NA	P	1.5" M/O
TH 113	4407-12	RP 10	2006	Poor	D	1.5" O/L on old AC
^TH 113	5413-10	RP 5	2006	Good	D	5" O/L on reclaimed AC
TH 210	1805-72	RP 118	2010	N/A	D	2" O/L on existing concrete
TH 210	1805-72	RP 118	2010	N/A	P	2" O/L on existing concrete
*TH 212	1017-12	N/A	2008	N/A	D	New BAB
*TH 212	1017-12	N/A	2008	N/A	P	New BAB
*Section unable to be surveyed ^Section surveyed, however mix design unavailable for analysis M/O = Mill and Overlays; O/L = Overlay ; BAB = Bituminous on Aggregate Base						

3.2.4 Section Highlights from Field Visits

The notes from the field visits as well as select pictures from the visits are summarized in this section. The following subsections present each highway project and also indicates the study sections that were established.

3.2.4.1 Trunk Highway 1 (SP 8821-103)

- Location: Northern Minnesota west of Cook

- Construction Year: 2008
- Construction Type(s):
    - 1-1/2 inch overlay on old asphalt concrete (poor performer)
    - 4 inch overlay on reclaimed asphalt concrete (good performer)
- Section Length: Nearly 21 miles
- Site Notes (poor performer):
    - Section Start: RP 235
    - 1-1/2 inch overlay on old asphalt
    - Very poor ride
    - Appears to have extensive amount of high severity cracking
    - Large amount of wheel path cracking
    - Alligator cracking prevalent
    - Severe centerline joint segregation
    - Areas of overlay have completely failed
    - Mix appears to be very dry

Figure 3.2: TH 1 poor performer-section start

Figure 3.3: TH 1 poor performer-overview

Figure 3.4: TH 1 poor performer-surface profile

Figure 3.5: TH 1 poor performer-typical crack configuration

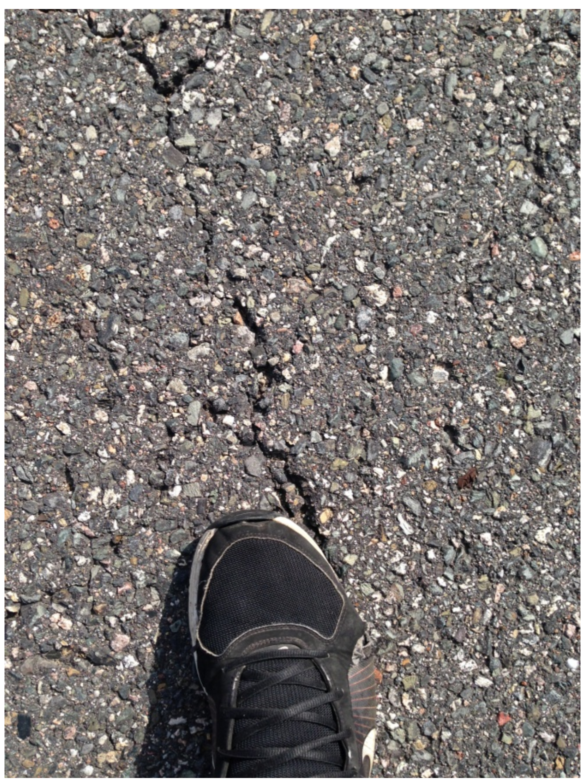

Figure 3.6: TH 1 poor performer-typical crack profile

Figure 3.7: TH 1 poor performer-overlay failure

- Site Notes (good performer):
  - Section Start: RP 230
  - 4 inch overlay on reclaimed asphalt concrete
  - Reclaimed section exhibits much smoother ride
  - Mix appears very dry
  - Significant amount of alligator cracking
  - Centerline joint segregation

Figure 3.8: TH 1 good performer-section start

Figure 3.9: TH 1 good performer-overview

Figure 3.10: TH 1 good performer-surface profile

Figure 3.11: TH 1 good performer-typical crack configuration

Figure 3.12: TH 1 good performer-typical crack profile

3.2.4.2 Trunk Highway 2 (SP 1102-59)

- Location: Northern Minnesota stretching through Bena
- Construction Year: 2003
- Construction Type(s):
  - 4 inch overlay on old asphalt concrete
  - Section Start: RP 157
- Section Length: Approximately 16 miles
- Site Notes:
  - Dry mix with a large amount of distributed cracking
  - Substantial fatigue and alligator cracking in wheel path
  - High amounts of medium to low severity transverse cracks
  - Centerline joint cracking throughout
  - Shoulder cracked both longitudinally and transversely throughout
  - Mix looks similar to TH 113

Figure 3.13: TH 2-section start

Figure 3.14: TH 2-overview

Figure 3.15: TH 2-surface profile

Figure 3.16: TH 2-typical crack configuration

Figure 3.17: TH 2-typical crack profile

3.2.4.3 Trunk Highway 6 (SP 3107-42)

- Location: North from Talmoon to the junction at TH 1
- Construction Year: 2004
- Construction Type(s):
    - 1-1/2 inch overlay on old asphalt concrete (poor performer)
    - 4-1/2 inch overlay on reclaimed asphalt concrete (good performer)
- Section Length: Nearly 19 miles
- Site Notes (Poor Performer):
    - Section Start: RP 118
    - 1-1/2 inch overlay on old asphalt concrete
    - Approximately 100 cracks per 1000 feet

Figure 3.18: TH 6 poor performer-section start

Figure 3.19: TH 6 poor performer-overview

Figure 3.20: TH 6 poor performer-surface profile

Figure 3.21: TH 6 poor performer-typical crack configuration

Figure 3.22: TH 6 poor performer-typical crack profile

- Site Notes (Good Performer):
    - Section Start: RP 123
    - 4-1/2 inch overlay on reclaimed asphalt concrete
    - Approximately 5 to 8 cracks per mile

Figure 3.23: TH 6 good performer-section start

Figure 3.24: TH 6 good performer-overview

Figure 3.25: TH 6 good performer-surface profile

Figure 3.26: TH 6 good performer: typical crack configuration

Figure 3.27: TH 6 good performer-typical crack profile

3.2.4.4 Trunk Highway 10 (SP 0502-95)

- Location: South of Little Falls, just outside Sartell
- Construction Year: 2005
- Construction Type(s):
  - 4 inch mill and overlay (good and poor performers)
  - Placed in two lifts 1-1/2 inch and 2-1/2 inch
  - Same mixture for both lifts
- Section Length: Slightly over 13 miles
- Site Notes (Poor Performer):
  - Section Start: RP 159

- Cracks recently sealed
- Inferior ride to RP 161

Figure 3.28: TH 10 poor performer-overview

Figure 3.29: TH 10 poor performer-typical crack configuration

Figure 3.30: TH 10 poor performer-typical crack profile and surface profile

- Site Notes (Good Performer):
    - Section Start: RP 161
    - Cracks are not sealed
    - Rides better than RP 159

Figure 3.31: TH 10 good performer-overview

Figure 3.32: TH 10 good performer-typical crack configuration

Figure 3.33: TH 10 good performer-typical crack profile and surface profile

3.2.4.5 Interstate 35 (SP 0283-26)

- Location: Section begins in Forest Lake and stretches south
- Construction Year: 2009
- Construction Type(s):
  - 4 inch mill and overlay on existing concrete
- Section Length: Approximately 8 miles
- Site Notes: (Based on drive through survey)

    - Four sections were surveyed, two in the northbound direction and two in the southbound direction
    - First section of northbound direction featured the greatest amount of cracking. Cracks were not full width, but were rougher than rest
    - Second section of northbound direction had relatively uniform crack spacing. Most cracks were full width across all three lanes.
    - First section of southbound showed the least amount of cracking, with all cracks being full width
    - Second section of southbound was very comparable to the second section of the northbound direction. Cracks were of relatively uniform spacing and full width.
- Due to the high traffic level of this roadway, no relevant pictures could be taken

3.2.4.6 Trunk Highway 53 (SP 8821-177)

- Location: North of Virginia
- Construction Year: 2008
- Construction Type(s):
    - 1-1/2 inch mill and overlay
- Section Length: 6 miles
- Section Start: Sign saying “TH 169 to Ely” (exit ¾ mile)
- Site Notes:
    - Moderate ride quality
    - Consistent amount of transverse cracking
    - Raveling in some locations
    - Shoulder cracking is not sealed
    - Cracks on primary driving areas sealed
    - Shoulder cracking 2:1 ratio in comparison to cracking in driving area

Figure 3.34: TH 53-section start

Figure 3.35: TH 53-overview

Figure 3.36: TH 53-surface profile

Figure 3.37: TH 53-typical crack configuration

Figure 3.38: TH 53-typical crack profile

3.2.4.7 Trunk Highway 113 (SP 4407-12)

- Location: Spans between Syre and Waubun
    - Project is split between two SP numbers
        - SP 4407-12 extends west from Waubun for approximately 6 miles
        - SP 5413-10 spans the remaining 9 miles to Syre
- Construction Year: 2006
- Construction Type(s):
    - 1-1/2 inch overlay on old asphalt concrete (poor performer)
    - 5 inch overlay on reclaimed asphalt concrete (good performer)
- Section Length: Slightly under 15 miles
- Site Notes (Poor Performer):
    - SP 4407-12
    - Section Start: RP 10
    - 1-1/2 inch overlay on old asphalt concrete
    - Near Waubun
    - Some transverse cracking meanders into longitudinal cracks
    - Potential reflective cracking

Figure 3.39: TH 113 poor performer-section start

Figure 3.40: TH 113 poor performer-overview

Figure 3.41: TH 113 poor performer-surface profile

Figure 3.42: TH 113 poor performer-typical crack configuration

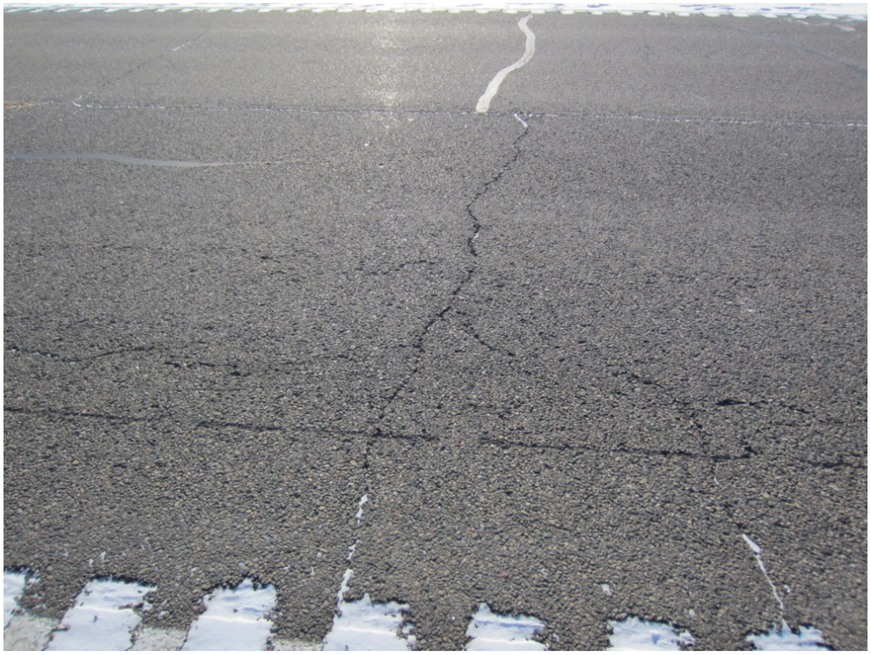

Figure 3.43: TH 113 poor performer-meandering transverse cracks

- Site Notes (Good Performer):
  - SP 5413-10
  - Section Start: RP 5
  - 5 inch mill and overlay on reclaimed asphalt
  - Near Syre
  - Good ride
  - Traditional transverse cracking

Figure 3.44: TH 113 good performer-section start

Figure 3.45: TH 113 good performer-overview

Figure 3.46: TH 113 good performer-surface profile

Figure 3.47: TH 113 good performer-typical crack configuration

Figure 3.48: TH 113 good performer-typical crack profile

3.2.4.8 Trunk Highway 210 (SP 1805-72)

- Location: Spans through Baxter
- Construction Year: 2010
- Construction Type(s):
    - 2 inch overlay on existing concrete
- Section Length: Roughly 4.5 miles

- Section Start: RP 118
- Site Notes:
    - Mix is quite coarse
    - Longitudinal joint is 100 percent cracked
    - Section exhibits transverse cracking roughly every 30 feet
    - 2 inch overlay over existing concrete
    - All transverse cracking is 100 percent reflective cracking
    - Raveling in various areas of the section

Figure 3.49: TH 210-section start

Figure 3.50: TH 210-surface profile

Figure 3.51: TH 210-raveling

Figure 3.52: TH 210-typical crack configuration

Figure 3.53: TH 210-typical crack profile

3.2.4.9 Trunk Highway 212 (SP 1017-12)

- Location: Spans through Chaska
- Construction Year: 2008
- Construction Type(s):
  - Bituminous over aggregate base
  - SMA mix design

- Section Length: Approximately 3 miles
- Due to high traffic levels, this site could not be surveyed
- Historical data on this section was available and will be presented in Chapter 4

3.3 Cracking Performance

Crack counts from site visits were combined with MnDOT's Pavement Management System (PMS) data to quantify cracking over the service life of the sections. The PMS data source contains all of the field performance (distress) data, specifically cracking performance of different pavement sections. Information pertaining to route types (Interstates, State highways, and US highways) and route numbers are included in this data source which contains 188 unique routes. The distress information includes transverse cracking, longitudinal cracking, rutting, raveling, patching, and longitudinal joint deterioration. Due to the main focus of this study pertaining to transverse cracking of asphalt pavements, transverse cracking was the only measure included in the analysis phase. The details on the statistical analysis of pavement cracking performance from PMS data against the mix design information was conducted in the task-1 of this study. The present task focuses on nine pavement projects and a total of 18 sections. The PMS data for these sections along with cracking performance from field visits is compiled and presented in this section. The task-3A report evaluates these performance measures against mix design parameters.

3.3.1 Cracking Performance Measures

The transverse cracking data in the PMS data is collected based on the severity of the cracks; low, medium and high. For each severity level the data is reported in terms of percent cracking (% cracking) which is calculated as 2 times the number of cracks per 500 feet length of the survey section. For purposes of conducting a statistical analysis between the amount of cracking and laboratory tests as well as asphalt mix parameters, a number of measures of field cracking performances can be calculated. In this study, the researchers looked at transverse amounts in terms of total cracking. This is the sum total of low, medium and high severity cracks.

The total cracking amounts for a given PMS section for each year of distress survey can be used to calculate additional cracking measures that are representative of field cracking performance. These measures for transverse cracking are described in Table 3.2. Please note that all data presented in this report as well as subsequent reports includes the crack counts that researchers collected during the site visits. Thus, the field visit information was incorporated with the PMS data providing the cracking performance information for the pavements from their construction until 2013/2014.

A graphical representation of the transverse cracking measures is displayed on Figure 3.54.

A. <u>Maximum Total Transverse Cracking Amount (MTCTotal):</u> this value is the absolute maximum transverse cracking amount experienced by the section, which is then normalized against the total number of years in service for the roadway. In this instance, 59 percent is the maximum amount of transverse cracking for the pavement over a service life of 11 years. This would result in a maximum total transverse cracking amount of 5.36 percent per year.

B. Maximum Total Transverse Cracking Rate (MTCRTotal): this is simply the greatest increase in transverse cracking between any two consecutive years. For example, trunk highway 2 exhibited a 12 percent increase in transverse cracking from the year of construction to the first year in service. Thus, 12 percent is the maximum total transverse cracking rate.

C. Average Total Transverse Cracking (ATCTotal): this particular measure is not explicitly defined on Figure 3.54. This value is the sum of all total transverse cracking measurements over the service life of the pavement divided by the total service life. Using the values from Figure 3.54, the calculation for average total transverse cracking is performed as follows:

a. $\text{ATCTotal} = \frac{12+19+26+27+28+28+28+33+38+49+59}{11} = 31.5 \text{ \% cracking/yr}$

Table 3.2: Description of Transverse Cracking Measures

Maximum Total Transverse Cracking Amount (MTCTotal)	Maximum transverse cracking amount (low + medium + high) of all survey years for a pavement section normalized against number of years for which pavement section has been in service.	% cracking/year
Maximum Total Transverse Cracking Rate (MTCRTotal)	Maximum increase in total transverse cracking amounts (low + medium + high) between any two consecutive years of service.	% cracking/year
Average Total Transverse Cracking (ATCTotal)	Sum of total transverse cracking (low + medium + high) for every survey year of a pavement section normalized against number of years for which pavement section has been in service.	% cracking/year

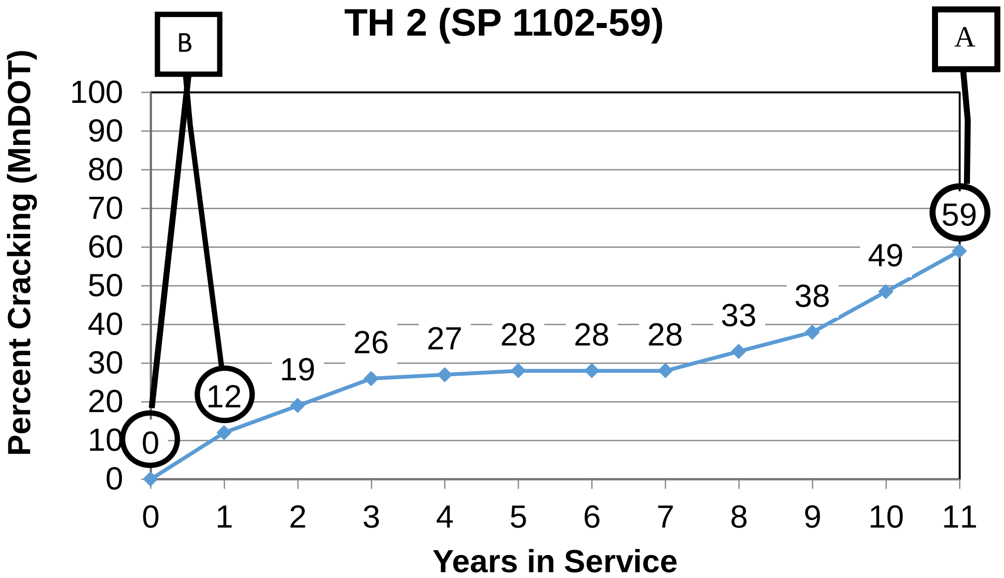

Figure 3.54: Example of Cracking Measures

The amounts of transverse cracking with respect to time for each of the roadways in this study are shown in Figure 3.55: Cracking Performance of TH 1 (SP 8821-103) through Figure 3.61. Note that there is no plot for I-35; this is due to the historical data for this section not being available at the time of this report submission.

3.3.2 Individual Roadway Cracking Performance

Trunk Highway 1 contained two pavement sections within the study domain (Figure 3.55). The section that is referenced as RP 230 was constructed with a 4 inch overlay on reclaimed asphalt while the RP 235 had a 1-1/2 inch overlay placed onto the old asphalt. The trend in the plot indicates that placing an overlay onto reclaimed asphalt showed a lower amount of cracking for a longer period of time.

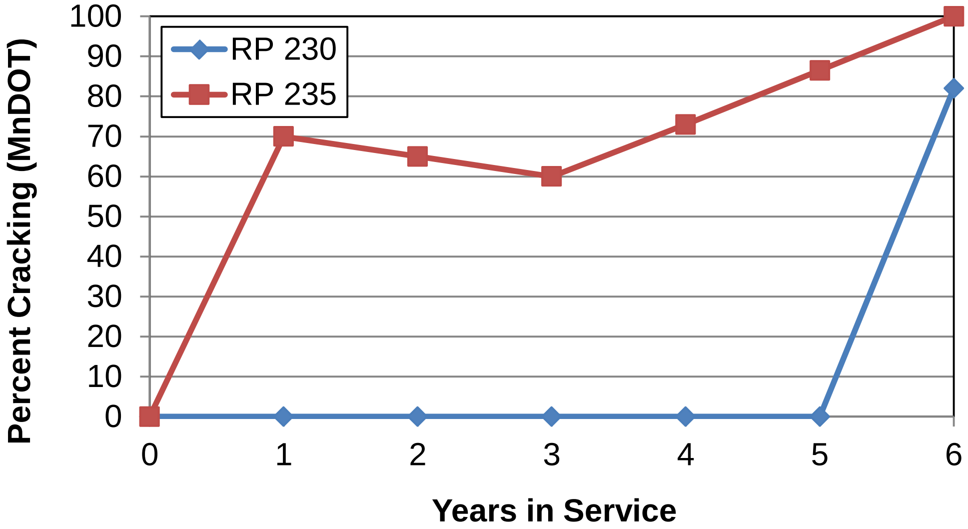

Figure 3.55: Cracking Performance of TH 1 (SP 8821-103)

The cracking performance of Trunk Highway 2 showed a gradual decline for eleven years. As the plot indicates, the deterioration of the roadway has been consistent over the last four years (Figure 3.56).

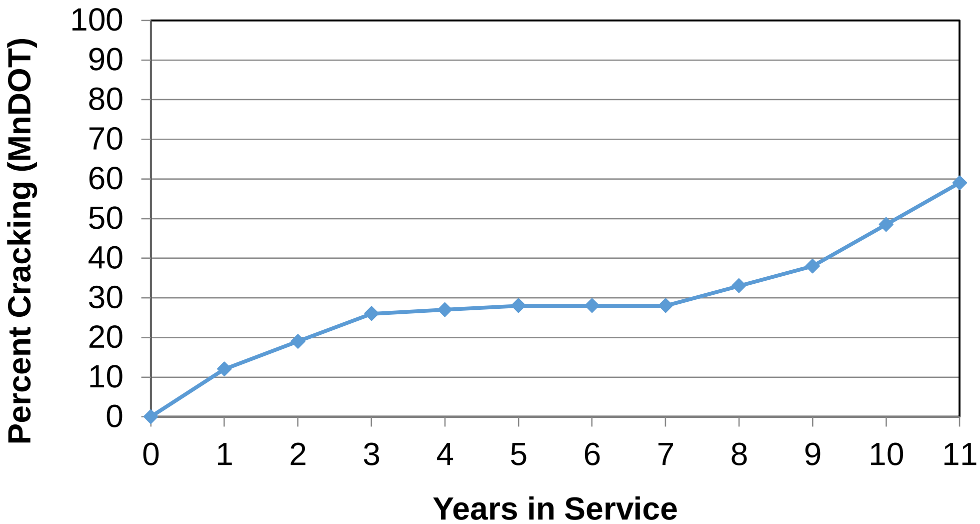

Figure 3.56: Cracking Performance of TH 2 (SP 1102-59)

The project for Trunk Highway 6 had two pavement sections associated with it. There is a noticeable difference in the performance of the two pavements (Figure 3.57). The section that started at RP 118 showed a large variation in cracking amounts, this may be due to the time of year or extreme temperatures when the automated crack counts were performed. This reinforces the need for site visits on periodic basis and to implement some form of consistency check for automated data collection. RP 118 was constructed with a 1-1/2 inch overlay placed on the existing asphalt. The portion referred to as RP 123, the better performer, was constructed with a 4-1/2 inch overlay on reclaimed asphalt.

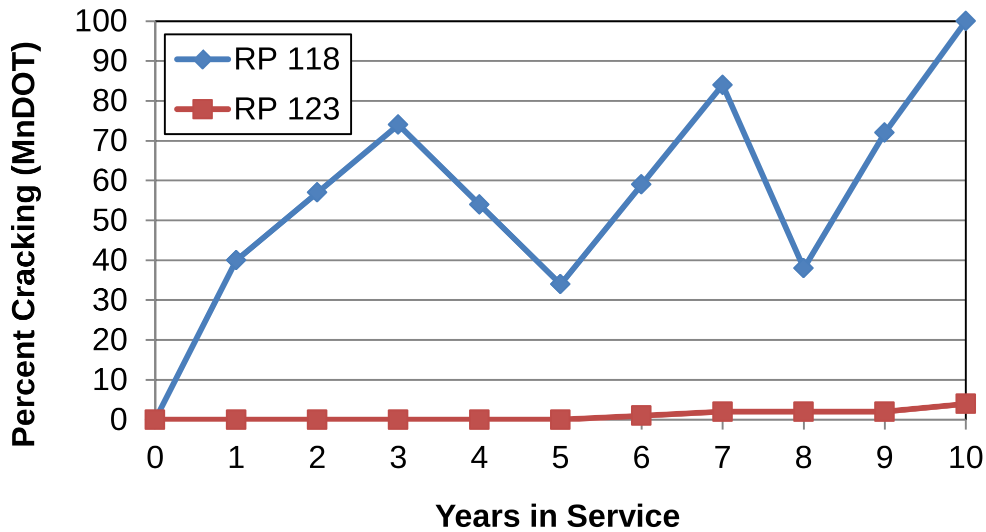

Figure 3.57: Cracking Performance of TH 6 (SP 3107-42)

The study area on Trunk Highway 10, a divided four lane highway, contained two different pavement sections. The cracking amounts are separated into driving lane and passing lane data (Figure 3.57). Both sections, RP 159 and RP 161, were constructed using a 4 inch mill and overlay. The cracks in the section beginning at RP 159 were sealed at the time of site visit where as for the section beginning at RP 161 the cracked were not sealed.

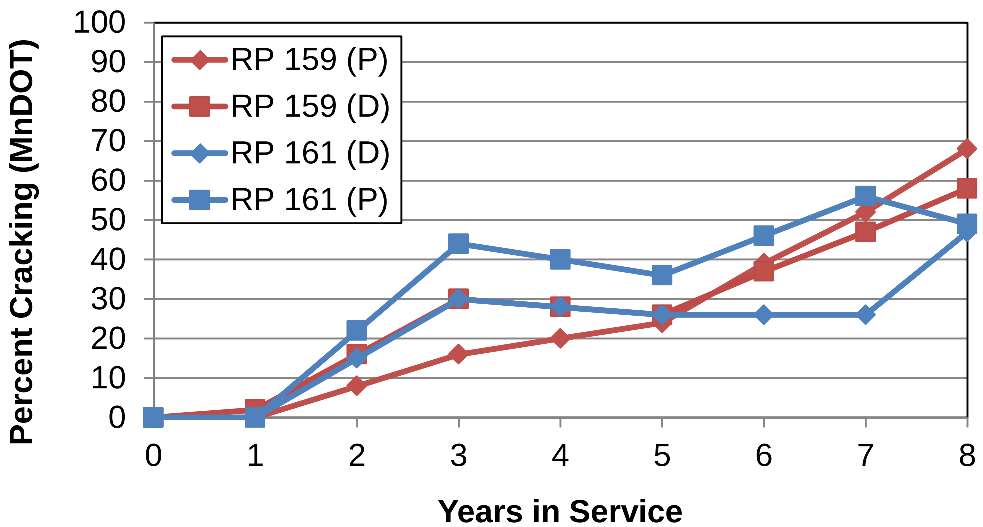

Figure 3.58: Cracking Performance of TH 10 (SP 0502-95)

Trunk Highway 53 was rehabilitated using a 1-1/2 inch mill and overlay. The cracking amounts vary greatly over time and also show a trend of increasing and decreasing (Figure 3.59), this is also inconsistency that was most likely resulted from automated crack counting system. Most of the cracking that was observed during the visual survey appeared to be reflective cracking. As with other section with asphalt overlay on PCC pavement majority of reflective cracking occurred during year 1 of service.

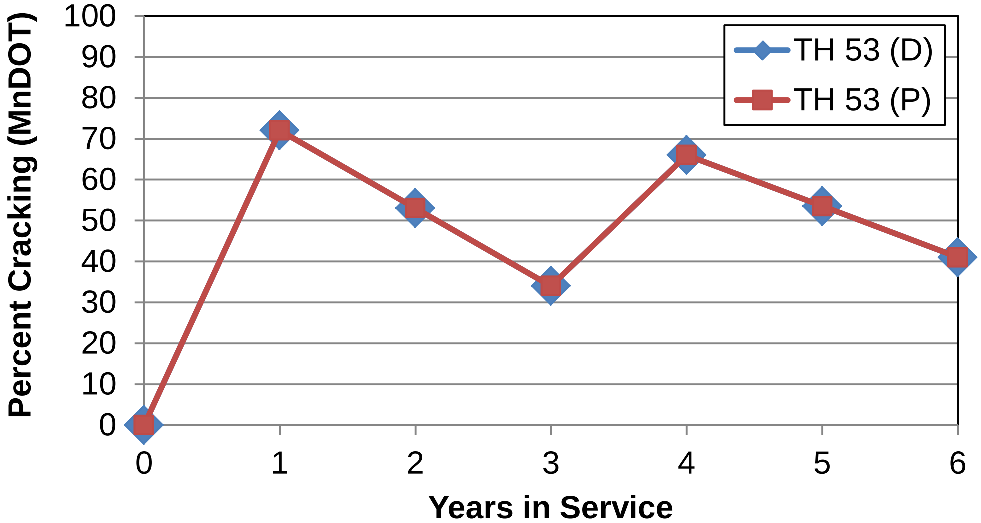

Figure 3.59: Cracking Performance of TH 53 (SP 8821-177)

The study area of Trunk Highway 113 also contained two differently constructed sections. RP 10 had a 1-1/2 inch overlay on existing pavement and RP 5 has a 5 inch overlay on reclaimed asphalt. Once again as with previous sections the overlay on reclaimed asphalt performed better than overlays on existing pavement. The plot shows that RP 10 started to crack in year one (Figure 3.60).

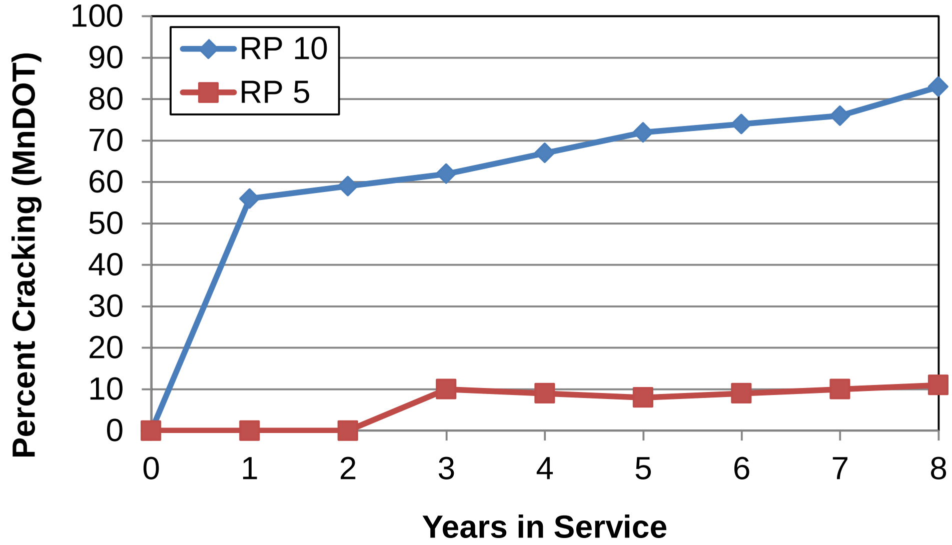

Figure 3.60: Cracking Performance of TH 113 (SP 4407-12)

A 2 inch overlay was placed on the existing Portland Cement Concrete pavement of Trunk Highway 210. The amount of cracking shown in Figure 3.61 indicates that the cracking is reflective from the underlying concrete. Another indicator of the reflective cracking is that the cracking amount shows little variation with time. Furthermore, the reflective cracks developed within 1 year of service. With approximately 30 foot joint spacing of PCC pavement the amount of transverse cracking comes to approximately 33%.

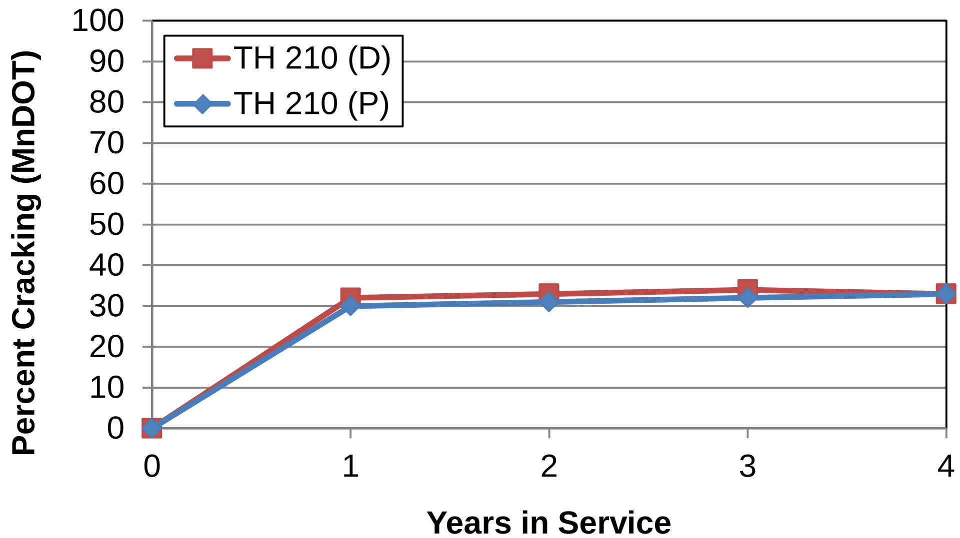

Figure 3.61: Cracking Performance of TH 210 (SP 1805-72)

The only pavement in the study group that is new bituminous on an aggregate base construction is Trunk Highway 212. Due to high traffic volume this section was not conducive to a walking visual survey. The data presented in Figure 3.62 is based on data from the automated crack counts from MnDOT. The section is constructed using SMA mixture and has shown excellent performance.

Figure 3.62: Cracking Performance of TH 212 (SP 1017-12)

3.3.3 Transverse Cracking Performance of All Study Sections

The transverse cracking performance of all pavement sections studied in this project is presented in this portion of the report. The performances are presented using the cracking performance measures described in Section 3.1 of this report. Please note that only the cracking performance data is presented herein, the analysis of data is presented in the Task-3A.

The maximum transverse cracking (MTCTotal) of each roadway per year is shown in Figure 3.63. As it can be seen from this plot the worst performing section (TH 1 RP 235) shows approximately 17% cracking per year of service, which translates into 100% cracking within six years of service. Of the pavement sections that were visited, TH6 RP123 demonstrates the best performance, with only 0.4% cracking per year of service.

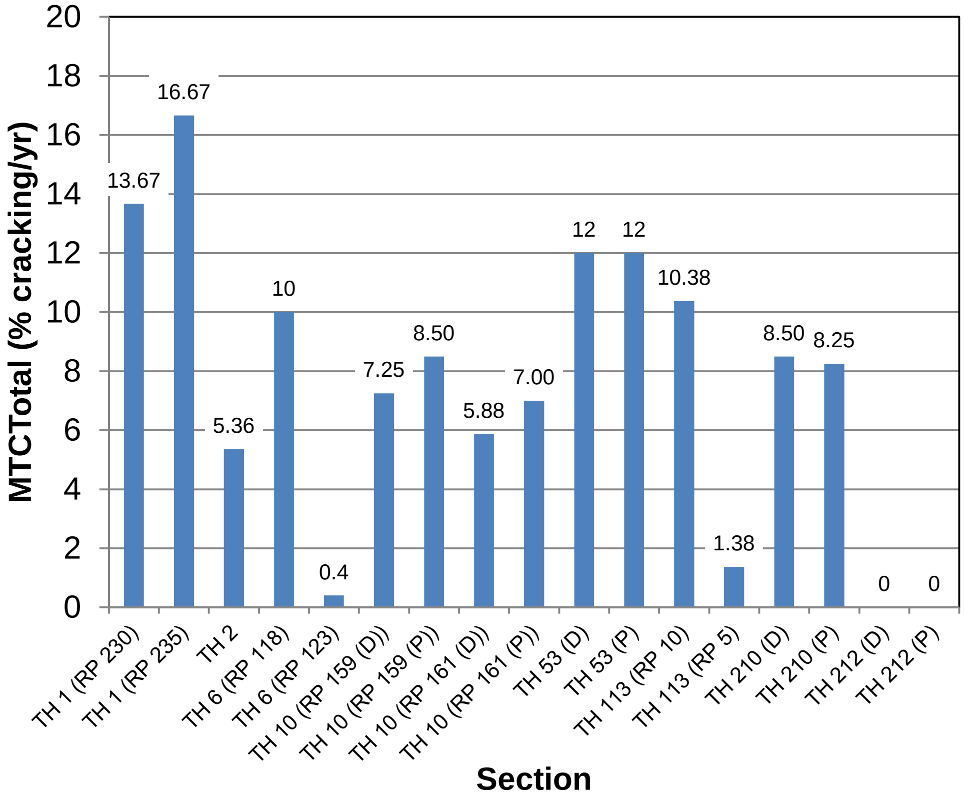

Figure 3.63: Maximum Transverse Cracking for All Study Sections

The maximum transverse cracking rate (MTCRTotal) data is presented in Figure 3.64. Once again this measure represents the maximum cracking increase that a pavement section experienced between two consecutive crack counts. Four of the study sections (TH 1 and TH 53 which are all located in Northern/North Eastern Minnesota) experienced relatively high cracking rates (between 70 and 82% cracking within a year). While more details are presented in Chapter 5 (Task-3A) of this report, in-general the overlay sections showed higher cracking rate early during the service, whereas reclaim sections showed the occurrence of high cracking rate later in the service.

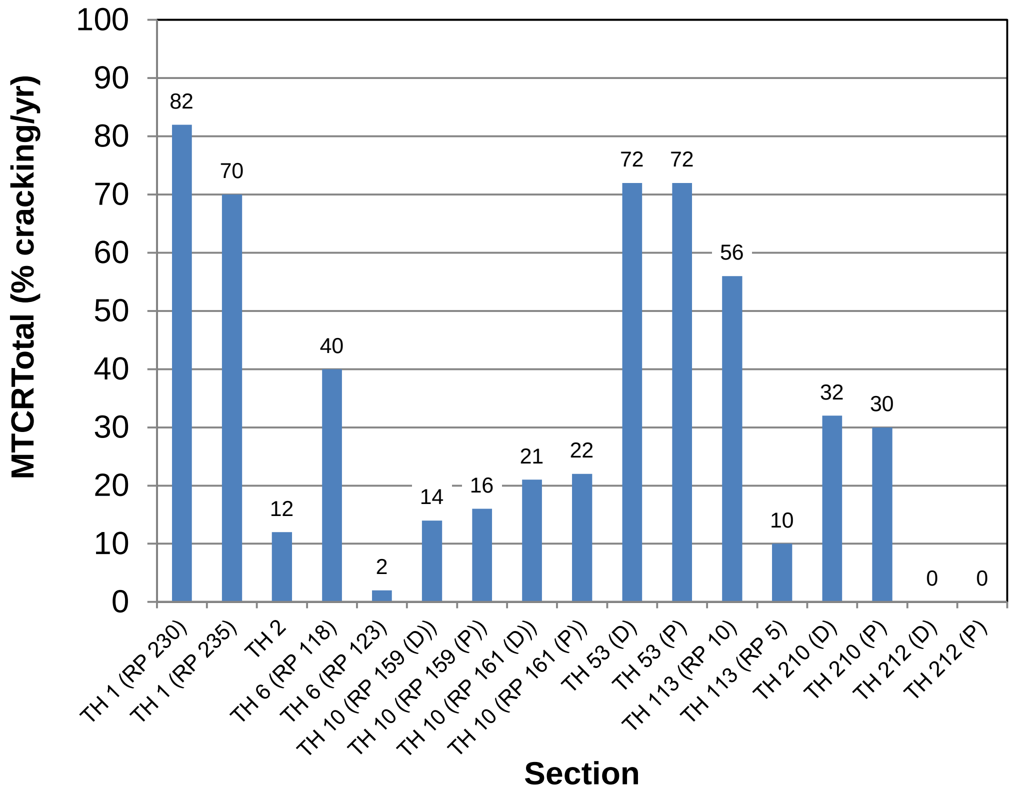

Figure 3.64: Maximum Transverse Cracking Rates for All Study Sections

The average transverse cracking (ATCTotal) information for all sections is presented in Figure 3.65. This measure differs from the previous measures in the sense that it accounts for cracking performance of the pavement section for each service year. Thus, this measure provides credit to pavements that have performed well for several years before cracking over a comparable section that displayed cracking within the first few years of service. The previous measures only focus on the maximum cracking amounts from all available data or maximum rate of cracking. With this measure the TH1 RP 230 and RP 235 sections show significantly different performance as the RP 235 experienced cracking early in the service life where as RP 230 experience cracking later in the life.

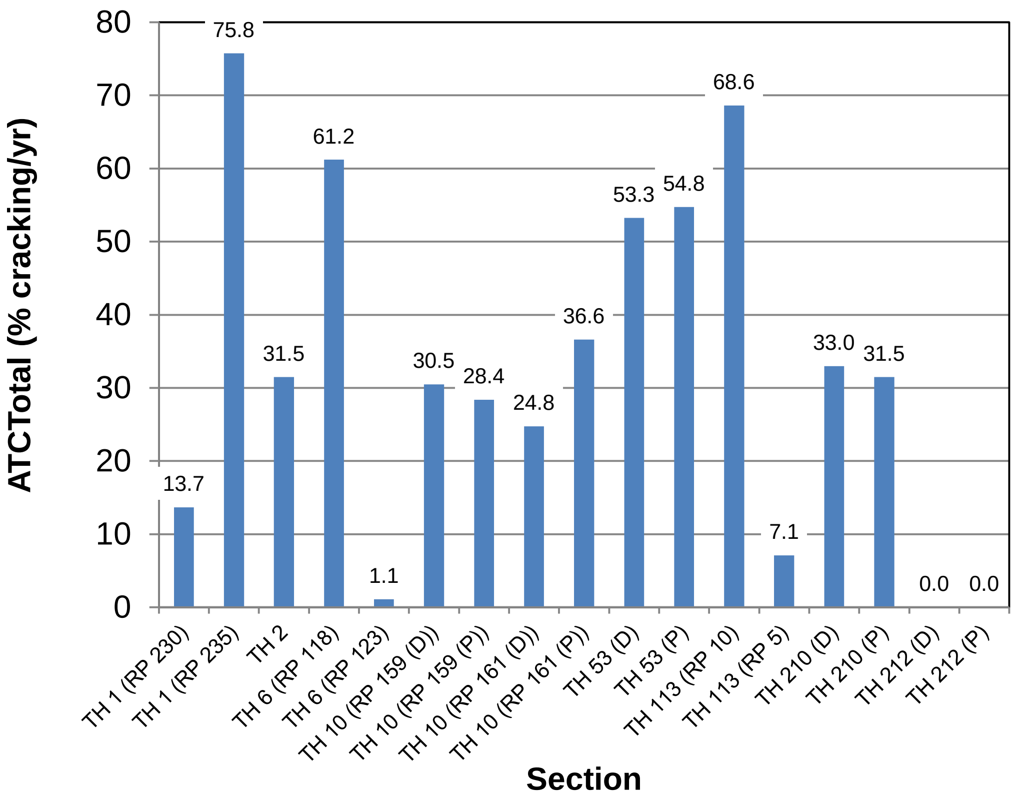

Figure 3.65: Average Transverse Cracking Amounts for All Study Sections

3.4 Summary

The Task-2A of this study focused on the field evaluation of nine highway sections. During this task the highway sections were visited and using a uniform site visit format a number of pavement study section were identified. The pavement study sections were evaluated to conduct crack counts as well as visual distress survey. The data collected during the site visits is summarized in Section 2.4 of this report, the raw crack count data is provided as appendix to this report. Furthermore, the locations for obtaining cored samples for performance testing were also identified. Using the sample collection information and on basis of the construction drawings, field sampling plans were developed and delivered to MnDOT staff. These plans are also attached to this report as appendix.

A number of cracking performance measures were developed through Task-1 of this project (Appendix-G). Those performance measures were utilized in conjunction with the pavement management data and information from field visits to quantify the cracking performance of pavement sections. The information collected and processed through this task is being utilized in Task-3A and 3B to make comparisons between field cracking performance and asphalt mix attributes as well as disk-shaped compact tension (DCT) fracture energy measurements.

While detailed analysis of the data is being conducted through Task-3A, some general observations from the cracking performance measures and sites visits are as follows:

- The average of the maximum cracking amount (MTCTotal) of all 18 study sections is approximately 7% per year of service. This information can be used to determine the number of years of service at which the pavement is expected to reach the state of 100% transverse cracking. On an averaged basis, using data from 18 pavement sections studied herein, approximately 14 years of service to reach 100% transverse cracking is obtained. The shortest life as seen from the study sections is expected to be 6 years.
- For the sections studied in this project the maximum cracking rate (MTCRTotal) is observed to be as high as 82% per year with average of 30.6% per year.
- The average of the average transverse cracking amounts (ATCTotal) for all 18 sections is approximately 30.7%. This measure indicates the average amount of cracking that would present on any section during the course of its service life.
- The asphalt layers on reclaimed sections show lower amount of cracking and delayed cracking as compared to mill and overlay sections on the same stretches of highways. It should be noted though that the reclaim sections consists of greater asphalt layer thicknesses (3 – 4 inch) as compared to mill and overlay sections (1-1/5 – 2-1/2 inch).
- The pavement sections consisting of asphalt overlay on PCC pavements showed significant reflective cracking within first year of service. Once all joint/cracks reflected into the overlay minimal additional cracking is observed.

CHAPTER 4: LABORATORY TESTING (TASK-2B)

4.1 Introduction

4.1.1 Overview of Task-2B

The Task-2B of the "Laboratory Performance Test for Asphalt Concrete" project involved laboratory testing of samples from several roadways across Minnesota. It is recommended that Task-2A be referred to for detailed background information on each site and individual cracking performance. The projects were chosen to obtain a wide cross-section of varying asphalt mixture designs and pavement structures. During the course of this task field samples were tested using the disk-shaped compact tension test (DCT). This report will provide a comparison of cracking performance and laboratory performance test results.

4.1.2 Organization of Chapter 4

It is recommended that this chapter be read in conjunction with "Chapter 5: Analysis of field performance data (task-3a) and Laboratory Testing Results (Task-3B)". This portion will describe laboratory testing methods and compare mix design parameters to laboratory testing results. All discussions relating mix design to laboratory testing will be covered in Chapter 5.4.

4.2 Overview of the Disk-Shaped Compact Tension (DCT) Test

4.2.1 DCT Test Description

The DCT test is standardized by ASTM D7313-13. The primary function of the test is to quantify the resistance an asphalt mixture will have to low temperature cracking. Low temperature cracking is the primary pavement distress in climates that experience extreme low temperatures and/or high rates of temperature drop. The discrete cracking of a material, as in the case of low temperature cracking, is a highly complicated phenomenon, and evaluation of the material beyond the linear response range helps close the gap between experimental results and actual field performance. All sections in this study, along with the majority of the State of Minnesota, undergo extensive low temperature climatic conditions. This study will use the DCT test on field cored samples to determine if any trends are found for use in future research projects.

Specimens for the DCT test can come from gyratory compacted pills or field cores. In the case of this study, all specimens came from field cored samples. Sample preparation involves sawing the pills or cores into 50 mm thick disks. Generally, both faces (top and bottom) of the disk are saw cut. The flat face, 25 mm diameter loading holes, and notch are then cut. See Figure 4.1 for a schematic of the typical DCT specimen and an actual prepared sample.

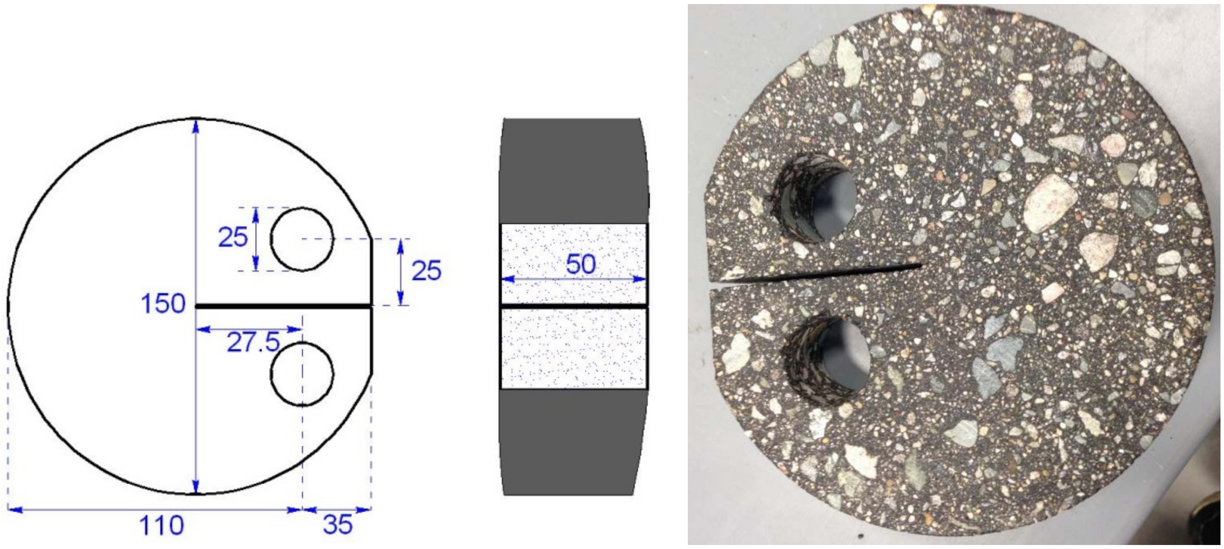

Figure 4.1: (a) Disk-shaped compact tension specimen geometry (dimensions in mm); (b) prepared DCT specimen

Prior to testing, gage points are first applied to the flat surface. These act as anchor points for the measuring device. This device is referred to as the crack mouth opening displacement (CMOD) gage. Once each specimen has been cut and gage points are attached, each specimen is carefully measured. Both the thickness and ligament length are vitally important to the accuracy of the results (see section DCT Results Description). The measurements (recorded by hand using a caliper) are averaged from several areas on the specimen to account for any variance in the specimen. The thickness is recorded at quarter points around the perimeter of the sample, and the ligament length (length between the inside of the notch and exterior edge of the sample) is recorded on both sides of the specimen. The averages of these results allow for the calculation of the sample area, the relevance of which will be explained later in this report. After completion of preparation and measurement, specimens must undergo temperature conditioning. DCT results are highly dependent on the temperature of the chamber. The ASTM specification for the DCT test (D7313-13) recommends conducting testing at a temperature 10°C greater than the low temperature performance grade of the binder in the asphalt mixture. While this may be applicable for quantifying the general resistance of a mixture to low temperature cracking, this value is not always indicative of the environmental temperature to which a mixture is exposed.

For this study, specimens were loaded into the testing chamber at a temperature 10°C greater than the 98% reliability environmental low temperature. For example, instead of testing a PG XX-34 at -24°C, temperature data shows (with 98% reliability) that this roadway will only experience -31°C. Therefore, DCT test conditioning for the corresponding specimens will target -21°C. This eliminates the unnecessary “penalization” for a binder in this scenario, as it will likely never see the extreme temperature recommended by the ASTM standard. Alternatively, a PG XX-28 binder tested at -18°C will not provide accurate DCT results for an environment experiencing temperatures colder than -28°C. Location is a primary function of this study. This required the research team to provide site-specific temperature conditioning data. In order to achieve this, historical temperature data was required to accurately predict this 98% reliability. LTPPBind was utilized to determine these values based on the specific location of each section.

Once the temperature has been determined and DCT specimens have been placed in the testing chamber, the DCT testing procedure can begin. The temperature conditioning process is the first step. In an effort to accurately model in-service conditions, the test temperature is achieved by ramping down the internal temperature of the chamber over a period of two hours. This is performed to avoid "shocking" the sample, as it is highly unlikely a roadway environmental temperature would drop from room temperature (≈20°C) to desired test temperature instantaneously. At the completion of the two hour ramping period, specimens are "soaked" at the target temperature for a minimum of two hours prior to the beginning of testing. Additional investigations on the impact of "shocking" samples and accuracy of the soaking period are ongoing.

A specimen is then mounted onto the testing apparatus Figure 4.2. As can be seen in Figure 4.2, pins are inserted into the two 25 mm loading holes. The pins facilitate the application of load via the loading clevis. The CMOD gage, as mentioned earlier, is clipped onto the gage points attached to the specimen (see Figure 4.3). The chamber is then allowed to cool back to the target temperature. At this time, the DCT testing can begin.

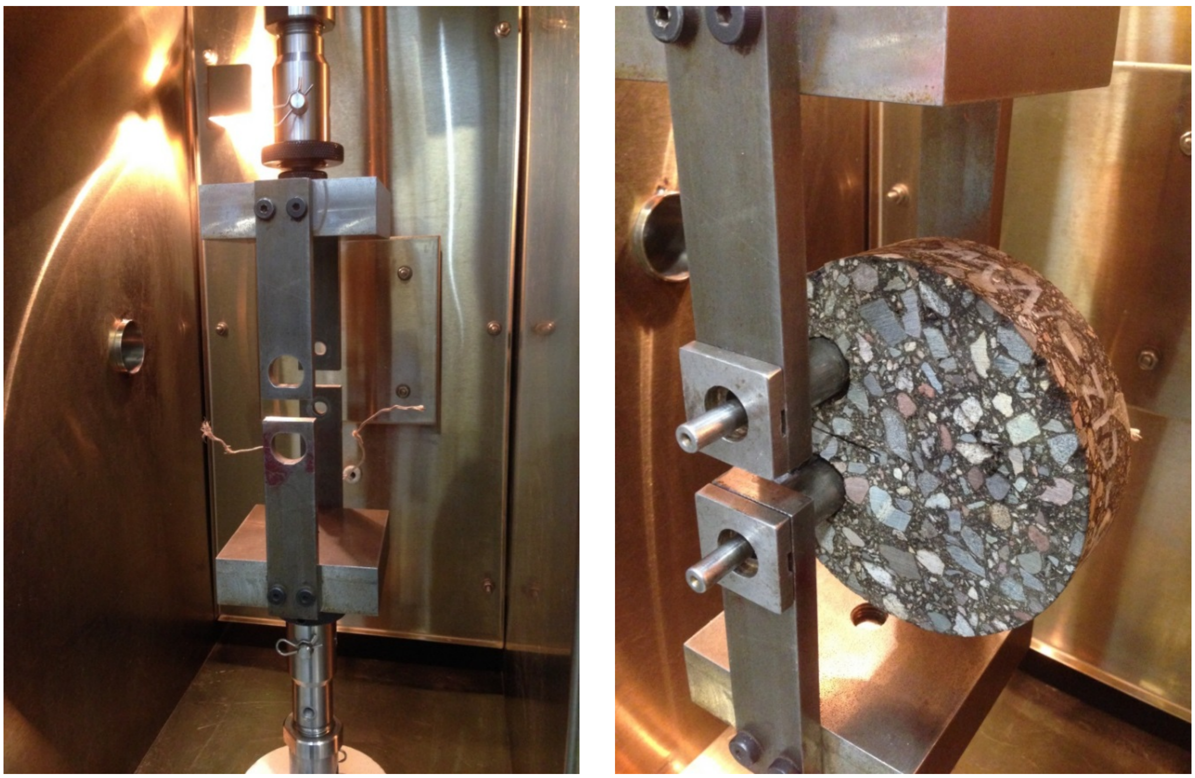

Figure 4.2: (a) DCT testing apparatus; (b) DCT specimen mounted onto apparatus

Figure 4.3: CMOD gage clipped to DCT specimen

A seating load of 100 N is applied prior to the beginning of testing. This reduces any potential loading "shock" that may cause a premature failure. Once preloading is complete, the upper loading clevis moves at a controlled rate; imposing a load on the DCT specimen. This rate is specified as 1 mm per minute and is dictated by the CMOD gage. The upper clevis increases the load applied to the sample, attempting to achieve this CMOD rate per minute. The load applied to the specimen will increase with little variation in the rate until initial failure of the sample takes place. Upon failure, the rate dramatically spikes. The upper clevis often has to retract slightly in order to keep the rate consistent with 1 mm per minute. The frame continues to add/remove load from the sample, until the load drops below 100 N (seating load value). At this time, the testing is stopped and the specimen is removed from the loading clevis. For this study, all samples from a section were tested during the same session. This reduced any potential variability from samples of the same section being tested on different days.

4.2.2 DCT Results Description

Fracture energy is the work required to fracture the DCT specimen normalized against the area of the specimen. Previous research has indicated that fracture energy of 400 J/m^2 is a desirable threshold for DCT testing. Mixtures testing above 400 J/m^2 exhibit little to no transverse cracking, while those below 400 J/m^2 feature higher levels of transverse cracking. In order to determine the fracture energy for each specimen, an extensive amount of information must be collected. The software controlling the rate at which the specimen is loaded stores 25 data points per minute. This data lists the time, load, CMOD displacement and chamber temperature over the course of each test. As mentioned earlier, the CMOD gage controls the rate of loading for the

DCT test. When coupled with the software, it serves the additional purpose of recording the displacement of the mouth opening over the time span of the test. The relationship between load and CMOD is the most important in terms of calculating the fracture energy of a specimen. Figure 4.4 illustrates a sample of the load versus CMOD plot. Note that CMOD and time are essentially a direct relationship; i.e. as CMOD increases, time generally increases. As can be seen during the initial portion of the test, the DCT specimen resists a significant amount of load while exhibiting very little crack mouth displacement.

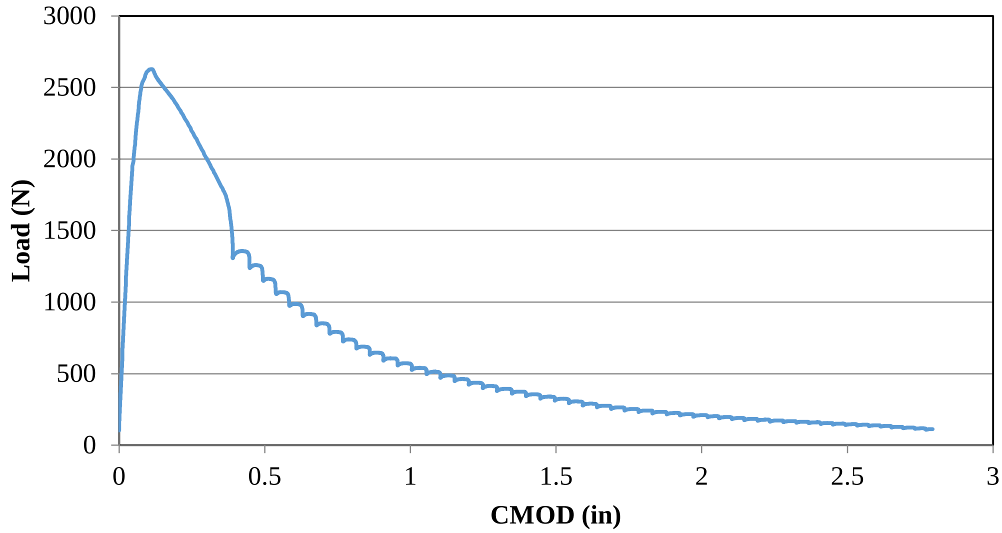

Figure 4.4: Sample DCT test output

At peak load the specimen experiences a quasi-brittle fracture, where a crack forms in a brittle manner at the inside of the notch of the DCT specimen. However, instead of forming a crack across the entire diameter of the specimen, softening can occur and the specimen continues to resist load as the crack mouth displacement increases. This phenomenon can be seen in Figure 4.4, as the load gradually decreases and CMOD continues to increase. Figure 4.5 best displays the behavior of softening on a microscopic level. This is essentially an instantaneous view of crack propagation across a sample. This area is known as the “cohesive zone”. On the right side, the crack has fully pulled the structure of the sample into two separate pieces. This is considered complete failure at this point, and the sample no longer is resisting any load. On the left, the crack is just beginning to damage the sample. Nearly full load resistance is still taking place in this region. Between these two points, various levels of damage and load resistance are underway. Generally, the greater a specimen can resist load after initial fracture and maintain longer durations of these “cohesive zones”, the greater the fracture energy.

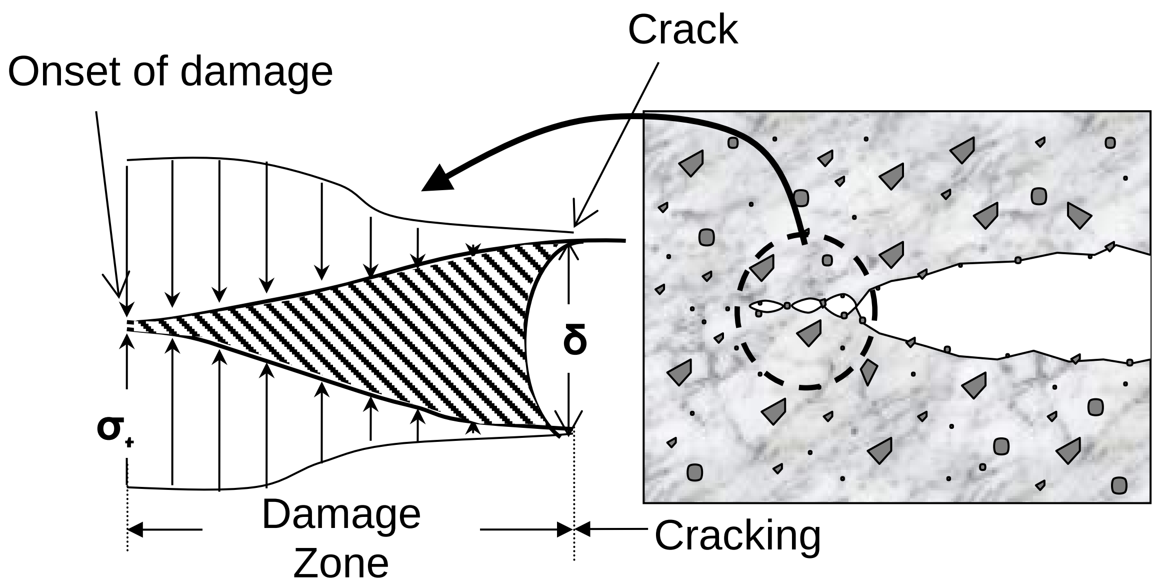

Figure 4.5: Schematic of cohesive zone

Figure 4.6 clarifies the discussion on the cohesive zone. Both the green and red areas represent two separate specimens each having the same area and exhibiting the same peak load, thus the same tensile strength. However, the green specimen behaves in a more ductile manner (the specimen exhibits a greater amount of softening) than the red sample. This results in a significantly higher fracture energy, as the fracture work is much greater for the green specimen as opposed to the red specimen.

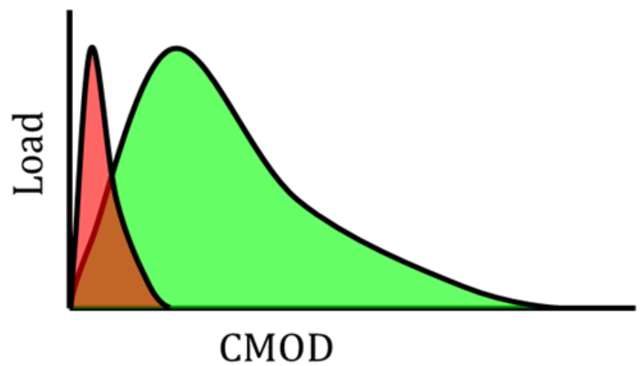

Figure 4.6: Comparison of brittle and ductile failure results

As mentioned before, an extensive number of data points are used to compile the plot shown in Figure 4.4. Using these discrete points, the area under the curve between any two consecutive points can be calculated. The sum of all of the individual areas is considered the <u>fracture work</u>. This work is then divided by the area of the fracture zone, which is the thickness of the sample multiplied by the ligament length. The result of this calculation is the <u>fracture energy</u> for an individual sample.

It should be noted that the normalization of samples for this study was especially important. Field cored specimens can vary greatly in thickness. This study incorporated three thin overlays (≈1.5” or 37.5 mm). While the ASTM specification advises all DCT specimens to have a 2” (50 mm) thickness, this is obviously not feasible for the thin overlay sections. As a result, thin overlays do not result in high fracture work values, but the normalization of the area allows for a fair comparison between the 1.5” (37.5 mm) disks and a typical 2” (50 mm) field cored sample.

4.2.3 Laboratory Testing Results

The laboratory testing results for each section will be presented in the following subsections. A summary of the site visits for various projects were previously discussed in section 3.2.3 and can be referenced from in Appendix-E (Table E-1).

Representative photos of specimens for each section can be found in Appendix-H. The subsections present results from each highway project and the individual study sections that were established.

Table 4.1 provides DCT results for each section in the study. This table includes the average fracture energies, standard deviations and coefficients of variance (COV) for each section from DCT testing of the field cores. Generally a COV of 15% is considered high variability for a set of samples. However, this only applies to a set of samples produced under controlled laboratory conditions. Being that the samples for this study are field cored, there are many potential variations that could have occurred from one sample to another; the most notable being a core coming from two different days of paving. The specimen thicknesses were relatively low due to thin lifts; this is anticipated to be primary reason for high COV in fracture energy results for the two sections denoted by asterisks (*) in Table 4.1. The COV values from the DCT testing for this study are provided primarily as a reference to show the potential variation in field cores. The values do not indicate any unnatural variability.

A notable challenge with this study is the influence of binder age. Each individual section will feature a variable (and unknown) amount of aging in the corresponding binder. An attempt to mitigate the impact of this factor is accounted for in the cracking measures by normalizing each measure over the service life. However, each binder does not necessarily age at the same rate. The sections all see different climatic conditions. Fracture energy will be influenced by the age of the binder, with brittle binders providing a lower fracture energy than a ductile binder. Therefore, the age of binder in field cores can have an unpredictable effect on fracture energy performance. This can lead to additional uncertainty when comparing fracture energy between sections.

Table 4.1: Summary of DCT testing results

Section	RP / Landmark	Performance	Test Temperature (°C)	Average Fracture Energy (J/m^2)	Standard Deviation (J/m^2)	Coefficient of Variance (COV)
TH 1	RP 235	Poor	-26.3	342	130	38%*
TH 1	RP 230	Good	-26.3	408	45	11%
TH 2	RP 157	N/A	-24.4	449	104	23%
TH 6	RP 118	Poor	-24.2	311	109	35%*
TH 6	RP 123	Good	-24.2	352	95	27%
TH 10	RP 159	Poor	-24.2	317	78	25%
TH 10	RP 161	Good	-24.2	365	66	18%
I-35	N/A	N/A	-20.8	379	50	13%
TH 53	169 to Ely	N/A	-25.7	397	130	38%
TH 113	RP 10	Poor	-23.7	182	17	9%
TH 113	RP 5	Good	-23.7	326	54	17%
TH 210	RP 118	N/A	-24.8	293	76	26%
TH 212	N/A	N/A	-20.7	1040	148	14%

TH 212 has an exceptionally high fracture energy. This could be due to a number of factors, including (but not limited to) one of the following: only new construction project in the study, higher quality binder (PG 70-34), or stone matrix asphalt (SMA) mix design. The relatively low COV for TH 212 also shows that this value is not likely an anomaly. The field cracking results from the Task 2A report also validate this number, as no transverse cracking has been observed on this section over the six year service life of the pavement.

TH 1 (RP 235) was a fairly poor performing section. It featured a high amount of transverse cracking throughout the service life, and currently exhibits 100% transverse cracking. The construction documents specify a 1.5 inch wear course for this section. However upon receiving field cores, the wear course was found to be approximately 1.25 inches and in very poor condition. Fabrication of DCT specimens was difficult, and two samples broke prematurely during testing. This section appears to have the potential for high fracture energy variability due to the poor quality of the roadway.

4.2.4 Comparison of Fracture Energy

The plots in this section provide visual comparisons between good and poor performing sections in terms of the average fracture energy of each section, as well as the results of individual test replicates (Figure 4.7 through Figure 4.15). The error bars in these figures are indicative of the standard deviation of the fracture energy measured from the individual replicates. Figure A-1 to Figure F-5 of Appendix-F illustrate the individual replicate for pavement sections where there are no comparison sections at a site, the average fracture energies for all sections are presented in Table 4.1. In all instances, the good performing section exhibited a higher average fracture energy than that of the poor performing section. This provides further validation that fracture energy can differentiate between inferior and superior performing sections on a roadway.

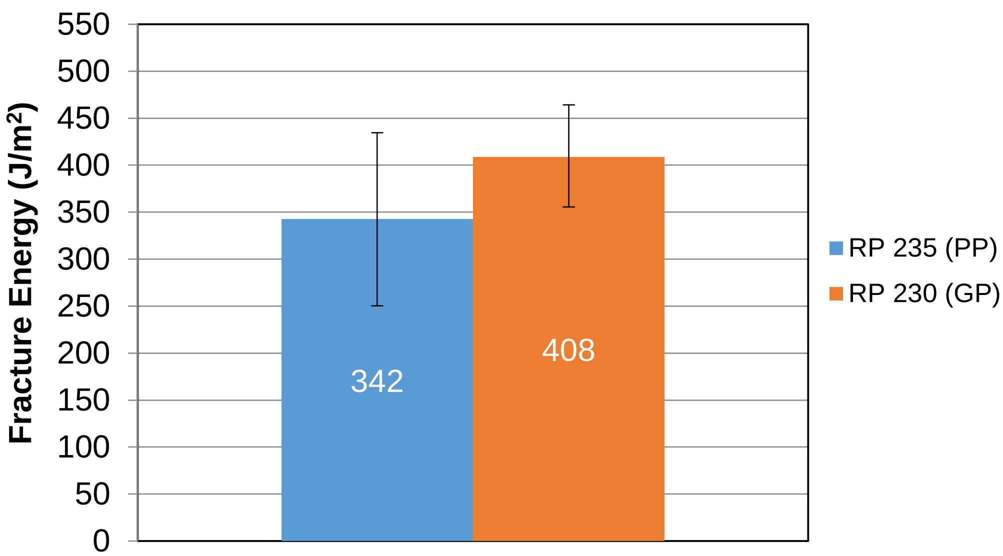

Figure 4.7: TH 1-poor performer versus good performer

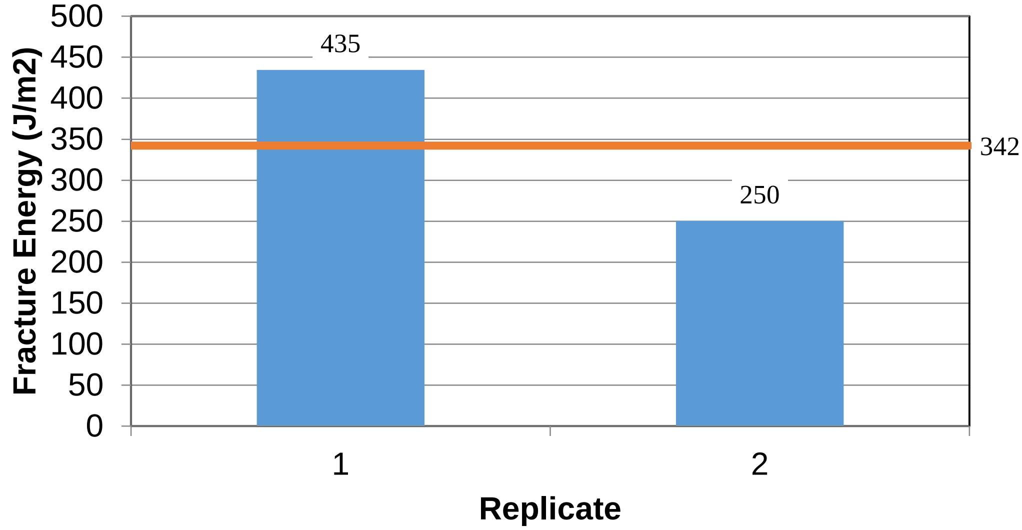

Figure 4.8: TH 1-RP 235 (poor performer) DCT results

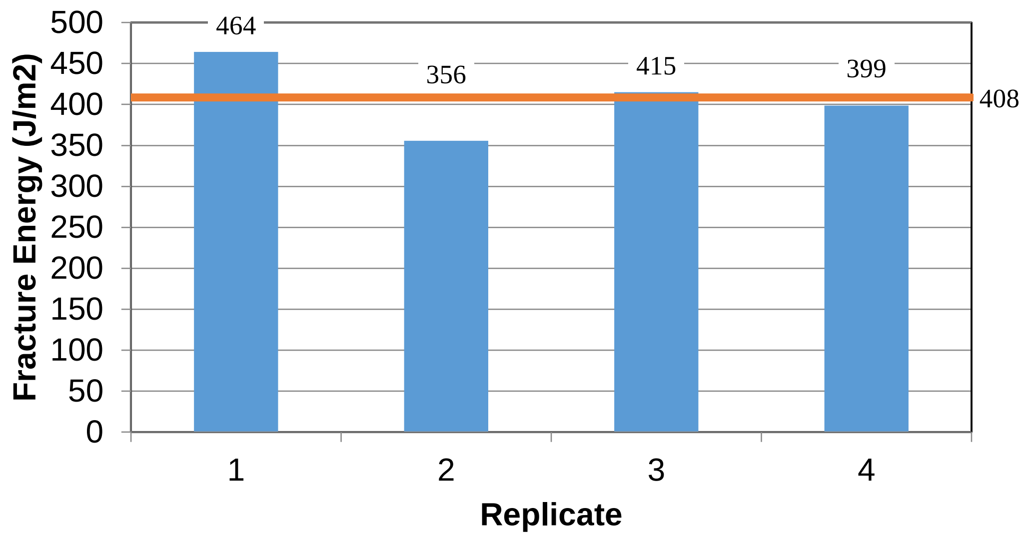

Figure 4.9: TH 1-RP 230 (good performer) DCT results

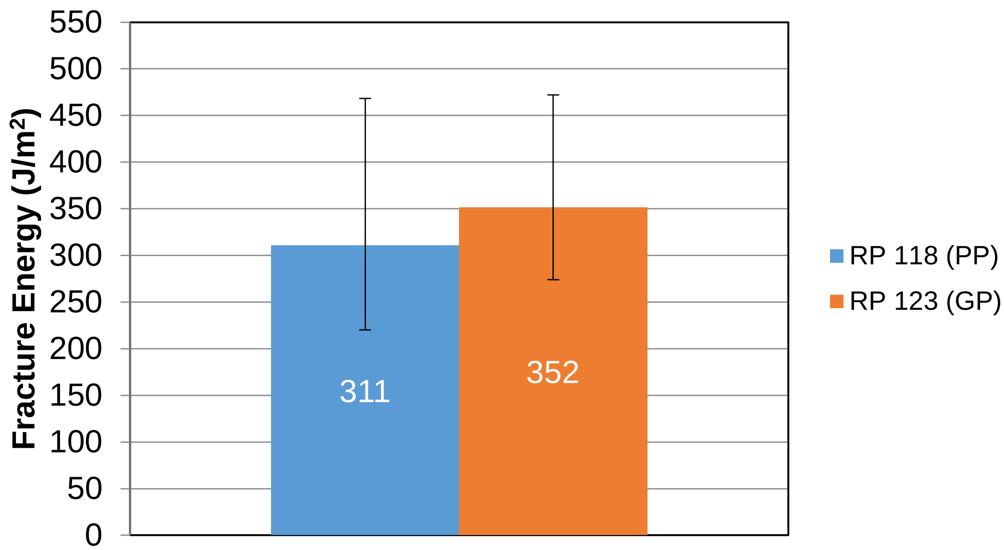

Figure 4.10: TH 6-poor performer versus good performer

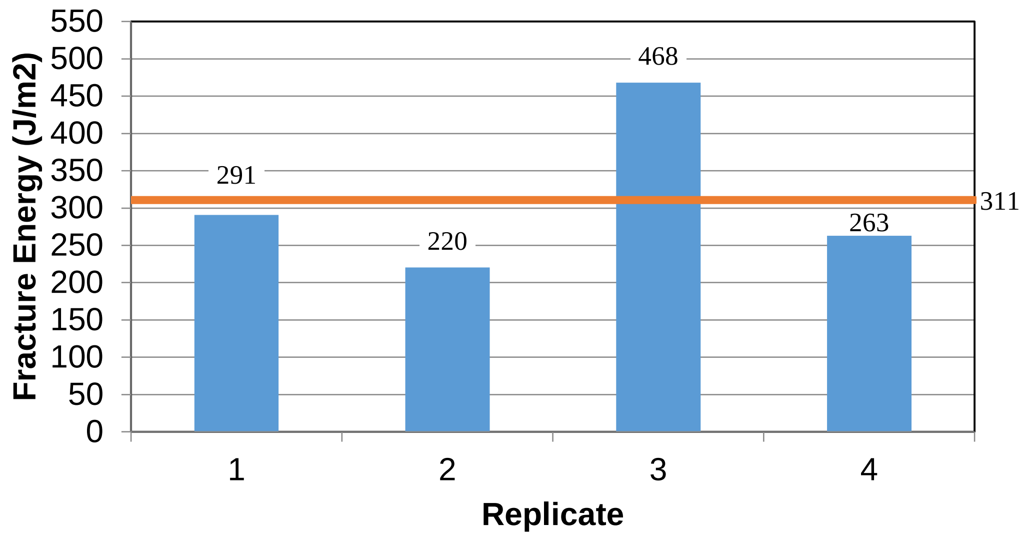

Figure 4.11: TH 6-RP 118 (poor performer) DCT results

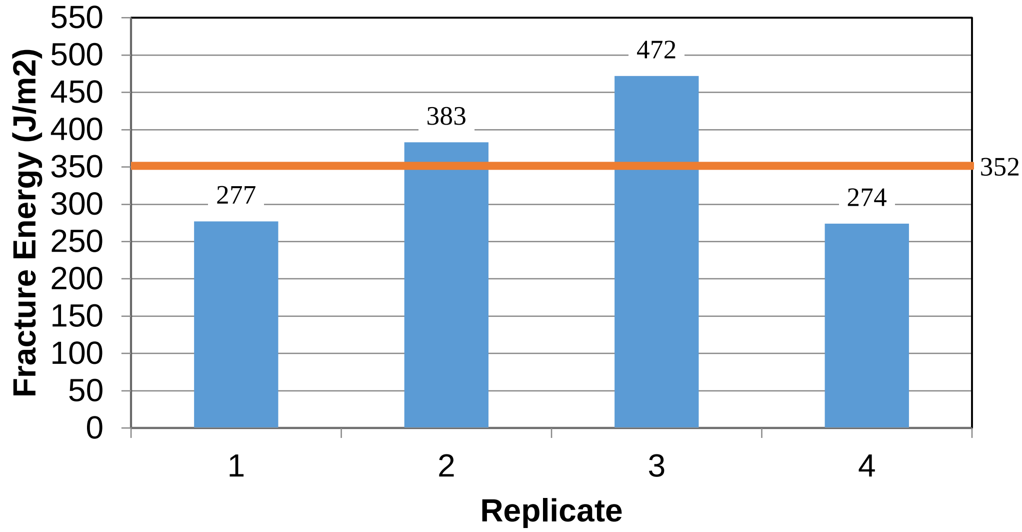

Figure 4.12: TH 6-RP 123 (good performer) DCT results

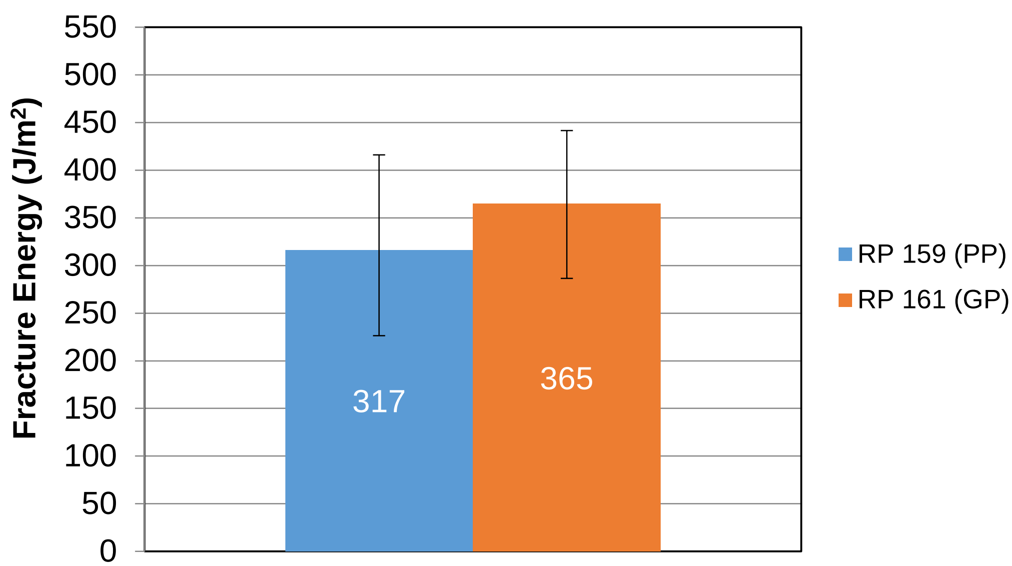

Figure 4.13: TH 10-poor performer versus good performer

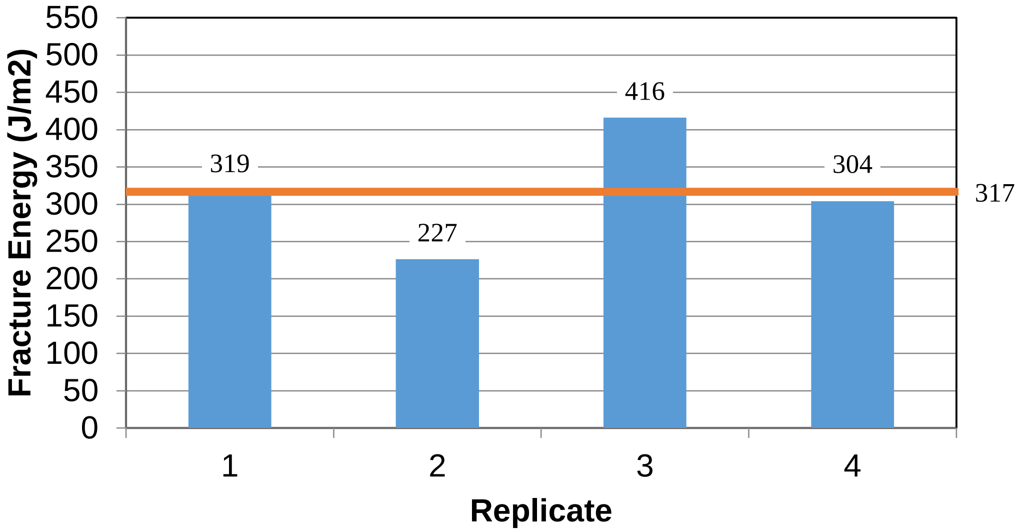

Figure 4.14: TH 10-RP 159 (poor performer) DCT results

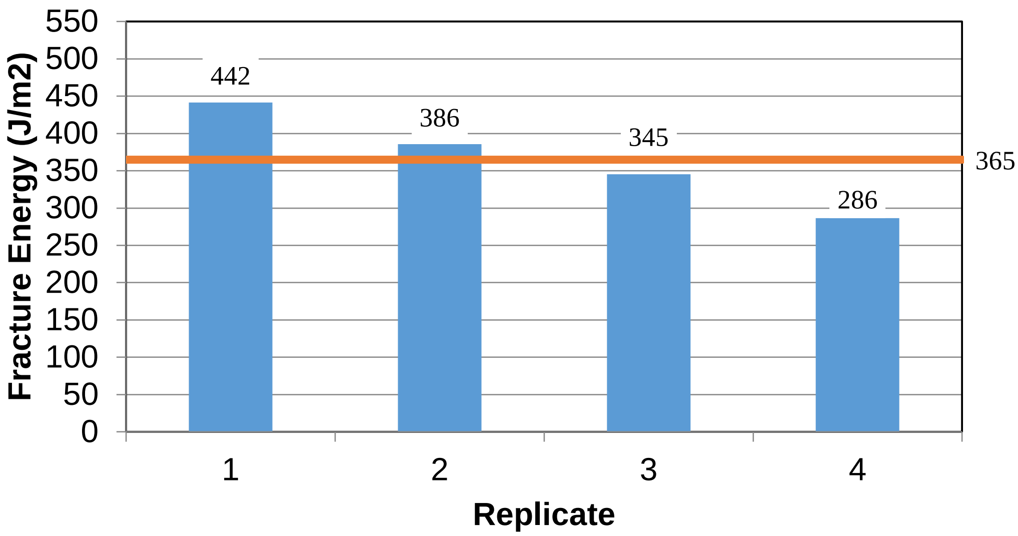

Figure 4.15: TH 10-RP 161 (good performer) DCT results

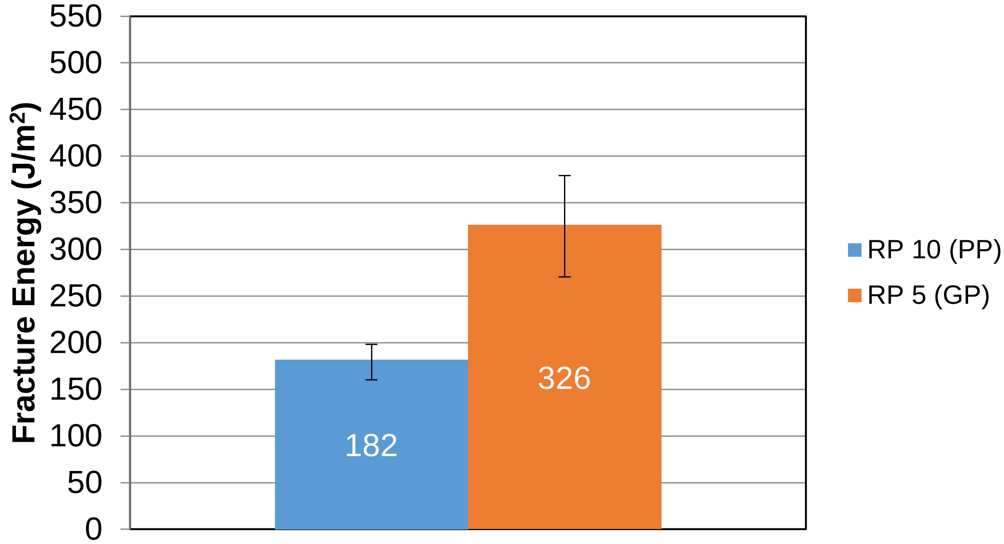

Figure 4.16: TH 113 poor performer versus good performer

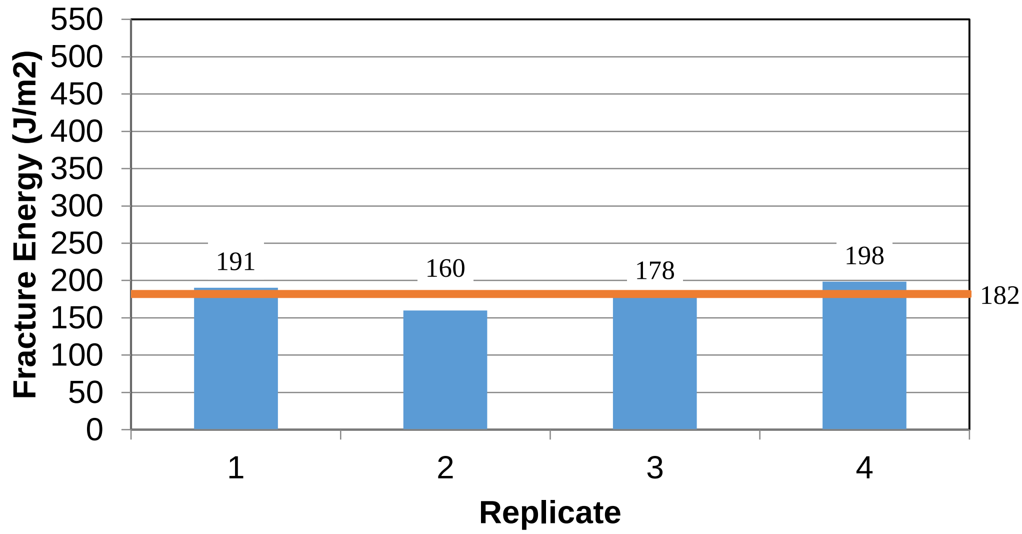

Figure 4.17: TH 113-RP 10 (poor performer) DCT results

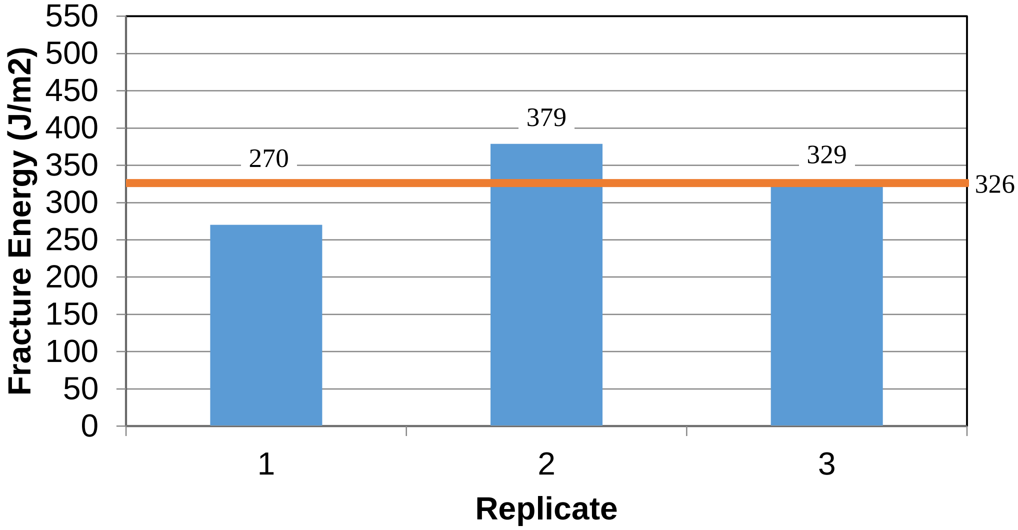

Figure 4.18: TH 113-RP 5 (good performer) DCT results

4.3 Comparison of Cracking Performance and Laboratory Testing

Crack counts from site visits were combined with MnDOT's Pavement Management System (PMS) data to quantify cracking over the service life of the sections. The PMS data source contains all of the field performance (distress) data, specifically cracking performance of different pavement sections. Information pertaining to route types (Interstates, State highways, and US highways) and route numbers are included in this data source which contains 188 unique routes. The distress information includes transverse cracking, longitudinal cracking, rutting, raveling, patching, and longitudinal joint deterioration. Due to the main focus of this study pertaining to transverse cracking of asphalt pavements, transverse cracking was the only measure included in the analysis phase. The details on the statistical analysis of pavement cracking performance from PMS data against the mix design information was conducted in the Task-1 of this study. The present task focuses on nine pavement projects and a total of 18 sections. The PMS data for these sections along with cracking performance from field visits is compiled and presented in this section. The Task-3A report evaluates these performance measures against mix design parameters.

4.3.1 Cracking Performance Measures

The transverse cracking data in the PMS data is collected based on the severity of the cracks; low, medium and high. For each severity level the data is reported in terms of percent cracking (% cracking) which is calculated as 2 times the number of cracks per 500 feet length of the survey section. For purposes of conducting a statistical analysis between the amount of cracking and laboratory tests as well as asphalt mix parameters, a number of measures of field cracking performances can be calculated. In this study, the researchers looked at transverse amounts in terms of total cracking. This is the sum total of low, medium and high severity cracks.

The total cracking amounts for a given PMS section for each year of distress survey can be used to calculate additional cracking measures that are representative of field cracking performance. These measures for transverse cracking are described in Table 4.2. Please note that all data presented in this report as well as subsequent tasks include the crack counts that researchers collected during the site visits. Thus, the field visit information was incorporated with the PMS data providing the cracking performance information for the pavements from their construction until 2013/2014. Please note that two additional cracking measures to the three previously described were used herein. The two additional measures include the Weighted Average Total Transverse Cracking (WATCTotal) and Total Transverse Cracking (TCTotal). These are presented in Table 4.2 along with the other cracking measures used and further described in the text succeeding the table.

Table 4.2: Description of transverse cracking measures

Maximum Total Transverse Cracking Amount (MTCTotal)	Maximum transverse cracking amount (low + medium + high) of all survey years for a pavement section normalized against number of years for which pavement section has been in service.	% cracking/year
Maximum Total Transverse Cracking Rate (MTCRTotal)	Maximum increase in total transverse cracking amounts (low + medium + high) between any two consecutive years of service.	% cracking/year
Average Total Transverse Cracking (ATCTotal)	Sum of total transverse cracking (low + medium + high) for every survey year of a pavement section normalized against number of years for which pavement section has been in service.	% cracking/year
Weighted Average Total Transverse Cracking (WATCTotal)	Total transverse cracking (low + medium + high) for every survey year of a pavement section is first normalized against the corresponding survey year. The sum of these values is then normalized against number of years for which pavement section has been in service.	% cracking/year/year
Total Transverse Cracking (TCTotal)	Sum of the total transverse cracking (low + medium + high) work over the service life. Total area is then normalized against the number of years for which pavement section has been in service.	% cracking

The primary function behind all five cracking measures is to determine a measure that accurate depicts the cracking performance for a section. A roadway experiencing 0% cracking for the first four years of the service life then cracking to a current amount of 50% clearly is a superior performer to a roadway cracking at 50% in year one and staying at 50% until the current time period. The five measures each portray the transverse cracking in a different fashion, so analyzing all five measures gives merit to each performance. An explanation of the transverse cracking measures can be found in the following bullets, along with a graphical representation on Figure 3.54.

A. Maximum Total Transverse Cracking Amount (MTCTotal): this value is the absolute maximum transverse cracking amount experienced by the section, which is then normalized against the total number of years in service for the roadway. In this instance, 59 percent is the maximum amount of transverse cracking for the pavement over a service life of 11 years. This would result in a maximum total transverse cracking amount of 5.36 percent per year.

B. Maximum Total Transverse Cracking Rate (MTCRTotal): this is simply the greatest increase in transverse cracking between any two consecutive years. For example, trunk highway 2 exhibited a 12 percent increase in transverse cracking from the year of construction to the first year in service. Thus, 12 percent is the maximum total transverse cracking rate.

C. Average Total Transverse Cracking (ATCTotal): this particular measure is not explicitly defined on Figure 3.54. This value is the sum of all total transverse cracking measurements over the service life of the pavement divided by the total service life. Using the values from Figure 3.54, the calculation for average total transverse cracking is performed as follows:

$$\text{ATCTotal} = \frac{12 + 19 + 26 + 27 + 28 + 28 + 28 + 33 + 38 + 49 + 59}{11}$$
$$= 31.5\ \%\ \text{cracking/yr}$$

D. Weighted Average Total Transverse Cracking (WATCTotal): this particular measure is not explicitly defined on Figure 3.54. This value is the sum of all total transverse cracking measurements first normalized against the individual survey years, then divided by the total service life. Using the values from Figure 3.54, the calculation for average total transverse cracking is performed as follows:

$$\text{WATCTotal} = \frac{\frac{12}{1} + \frac{19}{2} + \frac{26}{3} + \frac{27}{4} + \frac{28}{5} + \frac{28}{6} + \frac{28}{7} + \frac{33}{8} + \frac{38}{9} + \frac{49}{10} + \frac{59}{11}}{11}$$
$$= 6.3\ \%\ \text{cracking/yr/yr}$$

E. Total Transverse Cracking (TCTotal): this measure is best described in Figure 3.54. The value is the sum of the area under the percent cracking versus years in service curve (total cracking performance) divided by the total years in service. This measure offers a different perspective. While the other measures result in percent cracking per year, this measure quantifies the total amount of cracking a roadway experiences. For the values in Figure 3.54, 28.8 percent is the total transverse cracking amount.

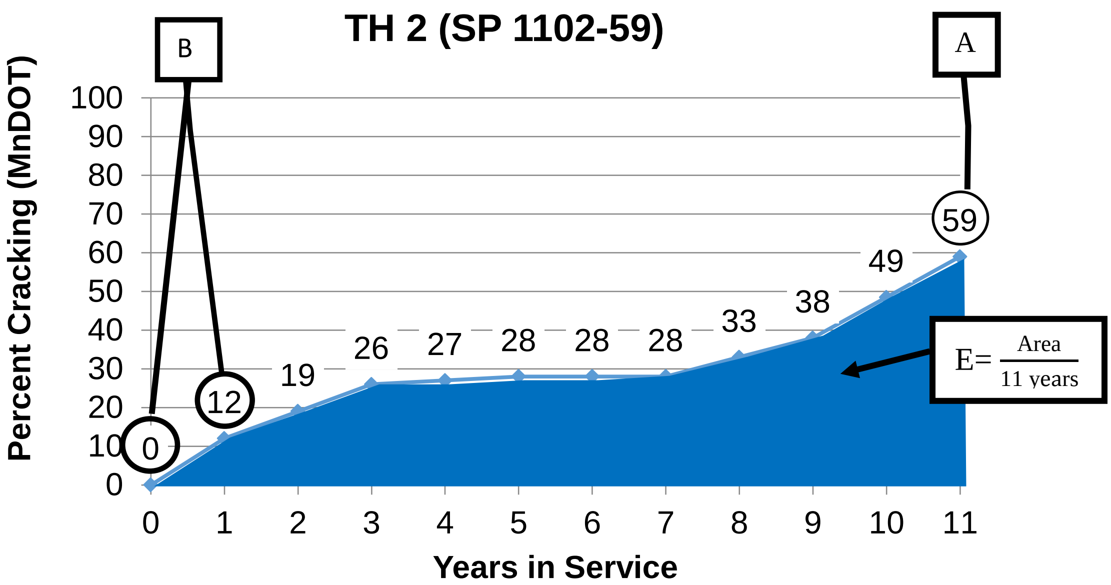

Figure 4.19: Example of Showing Different Cracking Measures

4.4 Transverse Cracking Performance of All Study Sections

Table 4.3 provides values of all cracking measures for each section. This information is provided in plots within the following subsections. Reference these sections for any discussions on the results provided.

Table 4.3: Summary of transverse cracking performance

Section	RP / Landmark	Performance	MTCTotal	MTCRTotal	ATCTotal	WATCTotal	TCTotal
TH 1	RP 235	Poor	16.7	70.0	75.8	29.1	67.5
TH 1	RP 230	Good	13.7	82.0	13.7	2.3	6.8
TH 2	RP 157	N/A	5.4	12.0	31.5	6.3	28.8
TH 6	RP 118	Poor	10.0	40.0	65.2	16.3	60.2
TH 6	RP 123	Good	0.4	2.0	1.1	0.1	0.9
TH 10	RP 159	Poor	7.9	14.3	29.5	5.9	25.6
TH 10	RP 161	Good	6.4	21.5	30.7	6.8	27.7
I-35	N/A	N/A	2.0	8.0	5.0	3.0	4.5
TH 53	169 to Ely	N/A	12.0	72.0	54.0	22.9	50.3
TH 113	RP 10	Poor	10.4	56.0	68.6	21.4	63.4
TH 113	RP 5	Good	1.4	10.0	7.1	1.4	6.4
TH 210	RP 118	N/A	8.4	31.0	32.3	13.8	28.1
TH 212	N/A	N/A	0.0	0.0	0.0	0.0	0.0

Figure 4.20 to Figure 4.24 illustrates the relationship between average fracture energy for each section and the various measures described in Section Cracking Performance Measures Each measure shows a correlation between decreasing fracture energy and increasing transverse cracking. While some of the trends are minor, note that the number of data points is relatively insufficient to determine solid relationships. Due to the variations between field cores (as mentioned earlier), it will take a significant amount of data to create trends that can be used as predictive functions. The trends in Figure 4.20 to Figure 4.24 are provided to suggest general guidance for future research. Considering this, the plots show encouraging trends for obvious reasons; the function of DCT testing is to correlate potential cracking amounts to a corresponding high or low fracture energy. For all measures, higher fracture energies result in lower cracking amounts.

Maximum total transverse cracking (MTCTotal) is a simplistic way to evaluate cracking performance. It does not apply any value to a roadway that performed at near 0% cracking for the majority of the service life. It is a quickly calculated measure that provides users with a general sense of roadway performance. Figure 4.20 illustrates the results for MTCTotal versus fracture energy.

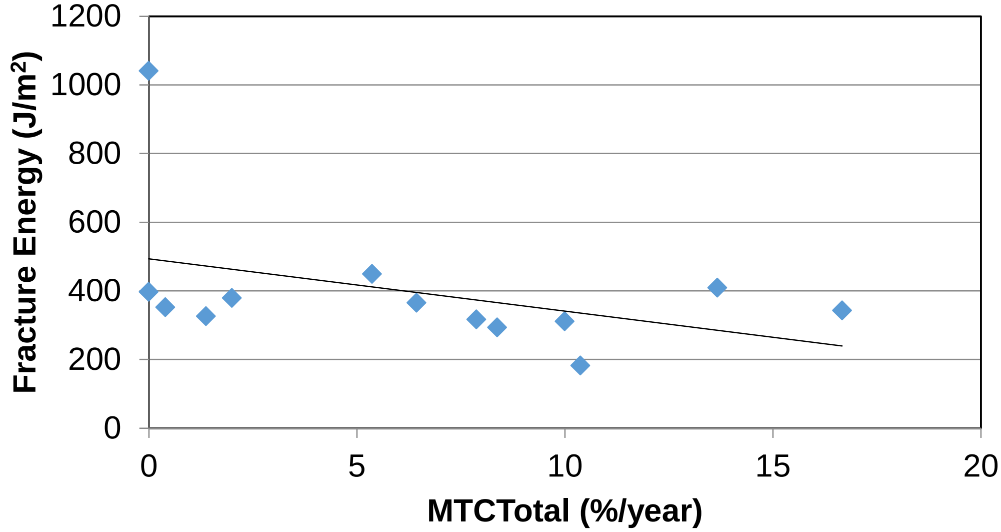

Figure 4.20: Fracture energy versus maximum total transverse cracking (FE vs. MTCTotal)

Maximum total transverse cracking rate (MTCRTotal) evaluates the maximum increase from two consecutive years. It provides a refined analysis, in comparison to MTCTotal, for a roadway. This is because gradual failure is generally more desirable than quick, drastic failure. Figure 4.21 shows results for MTCRTotal versus fracture energy.

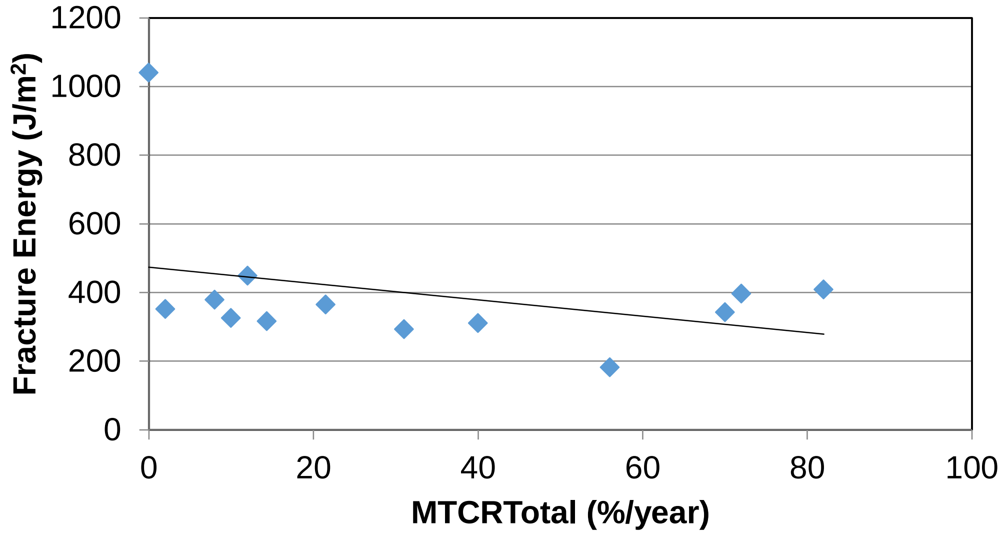

Figure 4.21: Fracture energy versus maximum total transverse cracking rate (FE vs. MTCRTotal)

Average total transverse cracking (ATCTotal) accounts for annual cracking rates, and is slightly more complex than MTCTotal and MTCRTotal, thus requiring more data to calculate. This is the first measure that takes into account annual cracking amounts. ATCTotal positively credits sections of roadways that exhibit lower levels of transverse cracking over the service life, and penalizes sections that crack early in service. Figure 4.22 illustrates results for ATCTotal versus fracture energy.

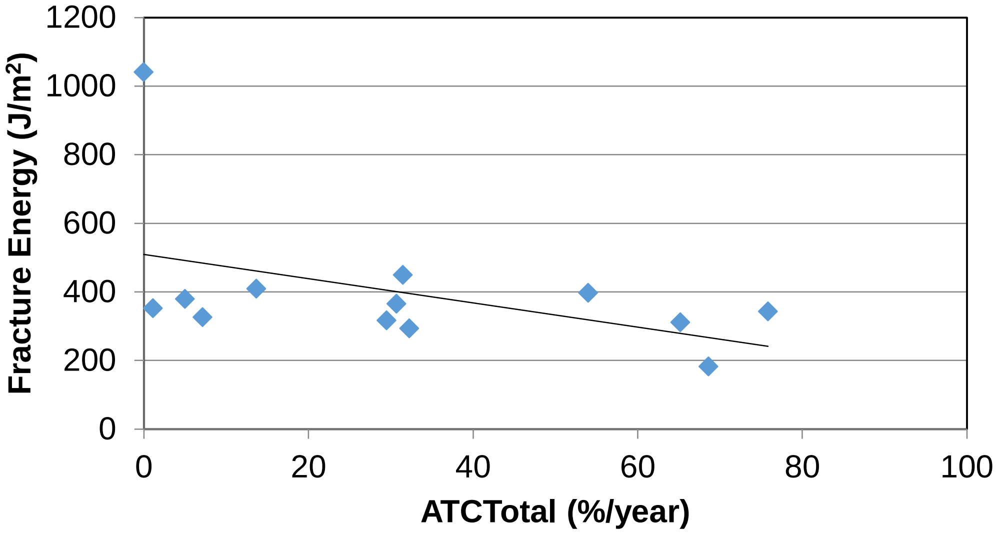

Figure 4.22: Fracture energy versus average total transverse cracking (FE vs. ATCTotal)

In comparison to ATCTotal, weighted average total transverse cracking (WATCTotal) provides further positive credit for sections that maintain low cracking levels throughout the service life. Building on the idea of evaluating annual cracking performance, each year is evaluated individually over the corresponding year(s) of service. The sum of these individual performance measures is then normalized for the total service life. WATCTotal versus fracture energy results can be seen in Figure 4.23.

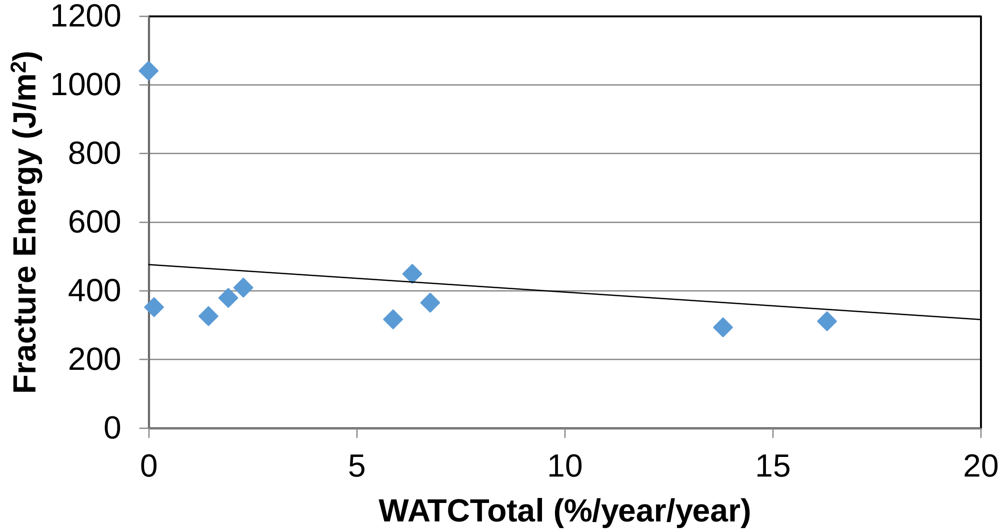

Figure 4.23: Fracture energy versus weighted average total transverse cracking (FE vs. WATCTotal)

Total transverse cracking (TCTotal) is the most complex measure presented in this report. Similar to the calculation of fracture energy, TCTotal is the sum of the transverse cracking performance exhibited by the roadway over the entire life of the pavement section (given by area under percent cracking and service life curve). This value is then normalized by dividing by the total service life. The results of TCTotal versus fracture energy can be found in Figure 4.24.

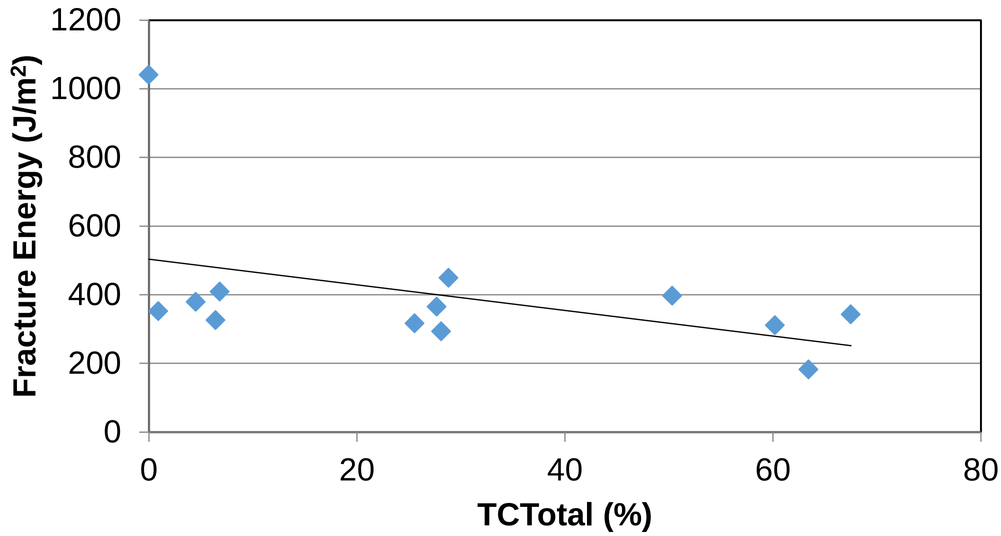

Figure 4.24: Fracture energy versus total transverse cracking (FE vs. TCTotal)

4.4.1 Normalization of Transverse Cracking Amounts against Traffic

Figure 4.25 illustrates a typical relationship between fracture energy and transverse cracking data normalized for traffic level. Total annual traffic, total annual truck traffic and daily traffic rates were all considered for this topic. All plots produced using this method resulted in a cluster of data near the origin and several points straying from this location, resulting in no true relationship. Upon removal of the "stray data", no relationship was found as the trend was essentially nonexistent. The data from this effort appears to validate that no strong correlation exists between traffic levels and fracture energy. Additional plots related to this topic can be found in an appendix to this report.

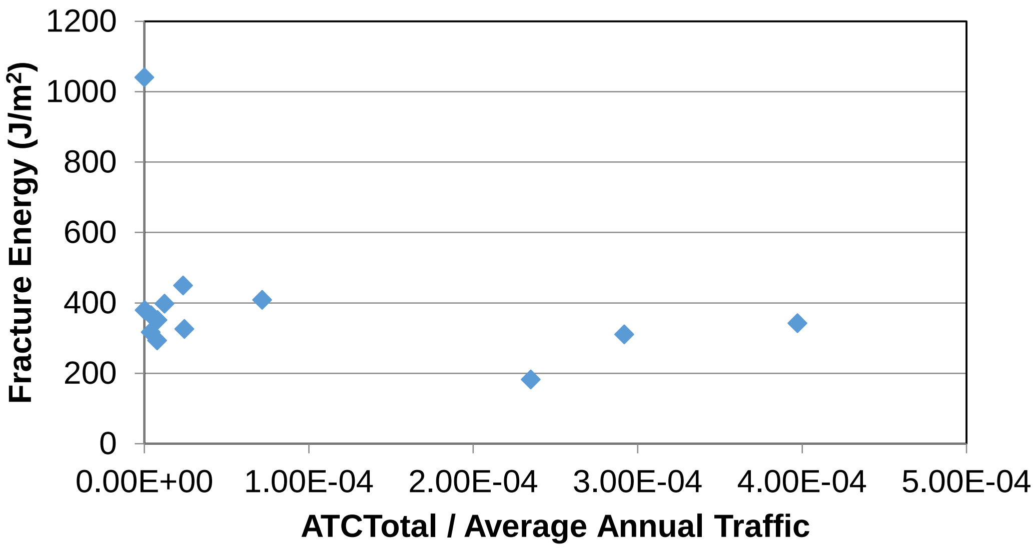

Figure 4.25: Fracture energy versus ATCTotal normalized for average annual traffic

4.5 Summary

The Task-2B of this study focused on the performance testing and comparison to field data for nine highways. During this task the field cores from each highway section, 13 sections in total, were tested using the disk-shaped compact tension test. The results of this effort are summarized throughout the various sections of this report. Data was compared to field performance using various transverse cracking measures, in an effort to reduce any reliability on a potentially misleading measure. These measures were developed through Task-1 of this project (c.f. Task-1 Report from June 2013) and modified as required during the analysis process. Those performance measures were utilized in conjunction with the pavement management data and information from field visits to quantify the cracking performance of pavement sections. This data is presented in several fashions, considering both traffic level of the section and pavement construction type.

The detailed analysis of the data conducted throughout Task-2B led to the following observations and potential recommendations for future research:

- A relationship between decreasing fracture energy and increasing transverse cracking amounts is apparent for various measures of cracking performance. This reaffirms the potential for using the DCT test as a performance indicator.
- Traffic levels do not appear to heavily influence cracking amounts.
- TH 212 performed at an exceptional level during testing, exhibiting an average fracture energy of 1,040 J/m^2. This is far greater than any other section in this study and well above the 400 J/m^2 threshold. Being that this section has experienced zero transverse cracking over the six year service life, it would appear to further validate the use of this threshold.
- This section is the only SMA mixture and new construction project in this study, making any comparisons with other mixtures practically impossible. There are a multitude of factors that could contribute to the success of this mix and these factors will continue to be monitored in future studies.
- While the small amount of data makes it difficult to confidently conclude any trend in this study., the testing of field samples provide additional validation for the DCT fracture test, cracking measures and contributing factors for fracture energy, but does not provide predictive function that can be used to determine extent of field cracking on basis of measured DCT fracture energy. Use of simulation models (such as, IlliTC) are recommended for that purpose.

This concludes the findings for the Task-2B of this study. Please reference the Task-3B report for findings related to mix volumetrics and fracture energy.

CHAPTER 5: ANALYSIS OF FIELD PERFORMANCE DATA (TASK-3A) AND LABORATORY TESTING RESULTS (TASK-3B)

5.1 Introduction

5.1.1 Introduction and Scope

This chapter provides the research activities conducted through Task-3A and Task-3B of the MnDOT contract 99008 work order 40 "Laboratory Performance Test for Asphalt Concrete." Task-1 of this study concluded that such a relationship may exist on basis of a statistical analysis of mix design records and pavement management data. The preceding phase of this study (Task-2A and 2B) was to identify several sites as candidates for further inspection and perform laboratory testing on field samples. The Task-2A portion of the report details these site surveys and establishes the basis for analysis in this report. The Task-2B portion of the report describes the laboratory testing procedure and results of this testing in comparison to field cracking performance.

The main purpose of the present tasks (3A and 3B) is to determine if any correlation exists between mix design properties and laboratory performance testing. Thus, Task-3A serves as a check or validation for the general findings made through Task-1 by use of select pavement sections and the reliable mix design as well as cracking performance data associated with those sections. By field evaluation of the pavement sections, Task-3A not only attempted to validate the findings from Task-1 it also presents conclusions regarding the question as to whether any of the mix parameters can be utilized as cracking performance prediction parameters.

The objective of Task-3B is to further validate the general findings make through Task-1 and Task-3A through fracture energy. By conducting disk-shaped compact tension (DCT) testing on field cored samples of which mix design properties are known, any preliminary correlations between performance testing and mix parameters can be observed. Since this study involves using actual field sections and testing of field procured materials, only a small number of sections could be tested. Therefore, any results presented herein should only be used to validate previous findings or for purposes of designing future research.

5.1.2 Overview of Task-3A and Task-3B

A summary of all the field sections included in this task are the same as Task-2A and 2B and can be referenced in Appendix-E. The Task-2A (Chapter 3) contains all of the detailed information about the sections and the surveys. Task-3A primarily focuses on the results of transverse cracking performance and Task-3B primarily focuses on the results from disk-shaped compact tension (DCT) testing. Refer to Chapter 4.2 (Task-2B) for a detailed description of DCT procedure and results.

Based on "Chapter 2: Analysis of Laboratory Test and Field Performance Data (Task-1)", several correlations have been established regarding the influence of mix design parameters on transverse cracking amounts. Based on the findings from previous studies as well as results from Task-2B of this study, the fracture energy of the mix affects the transverse cracking performance. Therefore as transverse cracking increases or decreases, fracture energy shows similar trends. A review of key findings from Task-1 that correlate mix design parameters to transverse cracking, and fracture energy, are as follows:

- A higher percentage of crack free pavements were represented by asphalt mixes that have lower adjusted asphalt film thickness (AFT) and higher voids in mineral aggregates (VMA). For pavements that have cracks present in them, neither adjusted AFT nor VMA showed consistent trends.
- Asphalt binder grade has a significant impact on the pavement cracking performance. Mixes containing -34 asphalt binders have a greater amount of crack-free pavements as compared to mixes containing -28 binders. A lower percentage of pavements with significant amounts of transverse cracking are represented by mixes with -34 binder grades as compared to those with -28 binder grades.
- The amount of asphalt binder has an effect on field cracking performance. The mixes with higher asphalt content showed lower amounts of cracking.
- Very few pavements constructed with all virgin materials were present during Task-1 analysis, thus limited data was available to draw any final conclusions regarding recycled materials.

These correlations provided guidance as to which mix design parameters required further inspection during this study. Using this information, the following mix design parameters were considered as potentially having an impact on field cracking performance:

- PG Grade
- PG Spread
- Asphalt Content
- Recycled Asphalt Content
- Voids in Mineral Aggregate (VMA)
- Voids Filled with Asphalt (VFA)
- Adjusted Asphalt Film Thickness (AFT)

Each of these parameters will be compared to fracture energy and transverse cracking performance measures for the aforementioned pavement sections. It should be noted that the objective here is not to develop correlations (or predictive equations) between mix design parameters and fracture energy but rather determine if any of the mix design parameters show potential for such correlations to be developed through future research. The cracking performance measures of interest are described in detail within section 3.3.1.

It is recognized that transverse cracking performance of certain mix design parameters may be altered by other factors. The initial review of data led to the conclusion that traffic level and asphalt layer thickness were two potentially significant variables. Effects of these variables was accounted for during the analysis procedure through normalization. Traffic level was taken into account by dividing the corresponding transverse cracking measure with the average daily truck traffic for each individual section. Similarly, asphalt layer thickness was normalized by multiplying the corresponding transverse cracking measure with the total asphalt layer thickness. Indirectly this also accounts for the potential added cost of a thicker asphalt layer.

After normalizing for either traffic level or asphalt layer thickness the resulting correlations between majority of cracking performance measures and asphalt mix parameters did not exhibit any recognizable trends as compared to before normalizing. Only selected plots with normalized data are provided in the subsequent section of this report for brevity, these are data sets where some observable trends were noticed. The remaining plots from this effort may be found in the Appendix-J and Appendix-K of this report.

5.2 Comparison of Mix Design Parameters with Transverse Cracking Performance of Pavement Sections (Task-3A)

5.2.1 Introduction

The asphalt mixture parameters were determined from the Mix Design Records (MDR) for each mix type obtained from the MnDOT Bituminous Engineering Office. The MDRs for each of the mixes are attached in Appendix-I. Sections 5.2.2 through 5.2.6 detail the analysis of the primary mix design parameters listed in section 5.1.2 with transverse cracking performance. In all of the following analysis procedures, Trunk Highway 212 is relegated to a separate series from the pooled section cracking data. This is due to the following factors: (1) the site was not visually surveyed, (2) a stone-matrix asphalt (SMA) mix is used; and, (3) the binder grade (PG 70-34) is the only one of its type in this study. It is shown in the analysis for completeness, but not considered influential to any of the trends mentioned in the following sections.

For any plots featuring a best fit regression line, it should be noted that the intention of this is not to show linear uniformity. The placement of this linear regression line is to simply show the approximate trend for the data being presented. In the data that follows, the blue markers represent all sections except for Trunk Highway 212 which is represented with a red marker. This is the typical condition unless explicitly defined in an alternative manner. In several instances multiple lanes (passing, driving etc.) were surveyed for same pavement section. Throughout this report the cracking performance for such sections are presented as average values for all lanes.

5.2.2 Effects of Asphalt Binder on Transverse Cracking Performance

5.2.2.1 PG Grade

Figure 5.1 to Figure 5.3 present the plot of PG grade against maximum total transverse cracking amount. These plots incorporate a box-and-whisker design, where the average is the “box” with the maximum and minimum values as the “whiskers”. The average values are for all pavement sections that were constructed using same grade of the virgin binder in the mix. The plots are generated in the order of decreasing averaged cracking measures with the PG grades. From these plots a loose trend of decreasing transverse cracking performance is observed as the binder grade goes in the order of PG 58-28, PG 58-34, PG 64-28 and PG 70-34.

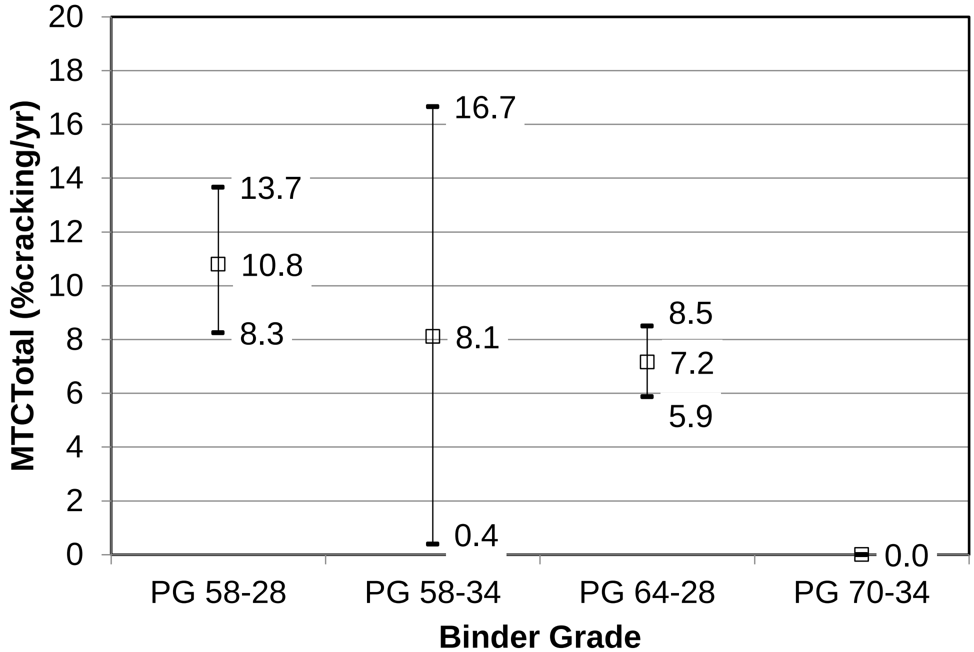

Figure 5.1: Effect of Asphalt Binder Grade on the Maximum Total Transverse Cracking Amount (MTCTotal)

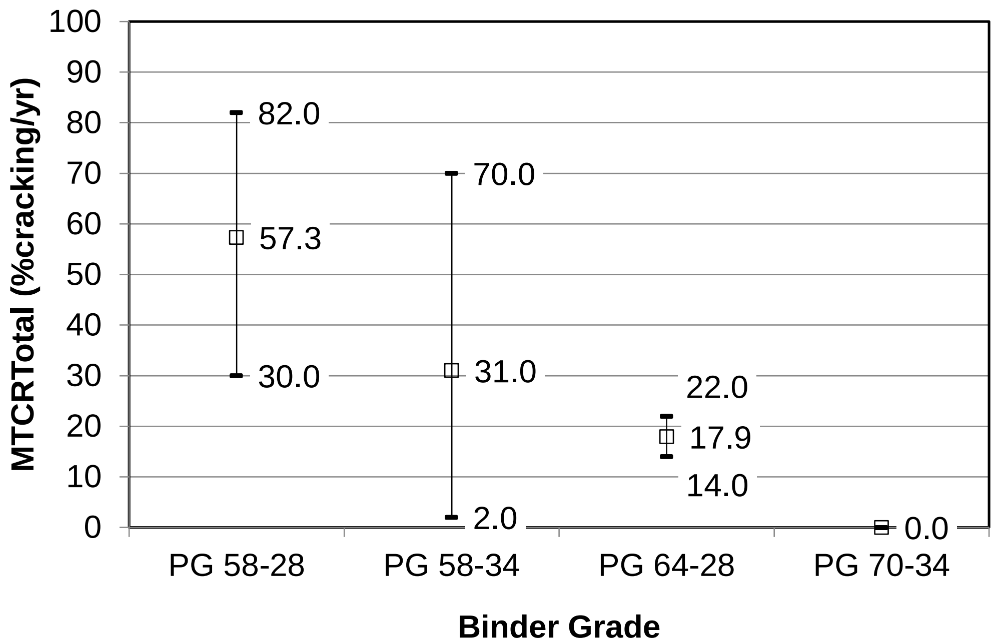

Figure 5.2: Effect of Asphalt Binder Grade on the Maximum Total Transverse Cracking Rate (MTCRTotal)

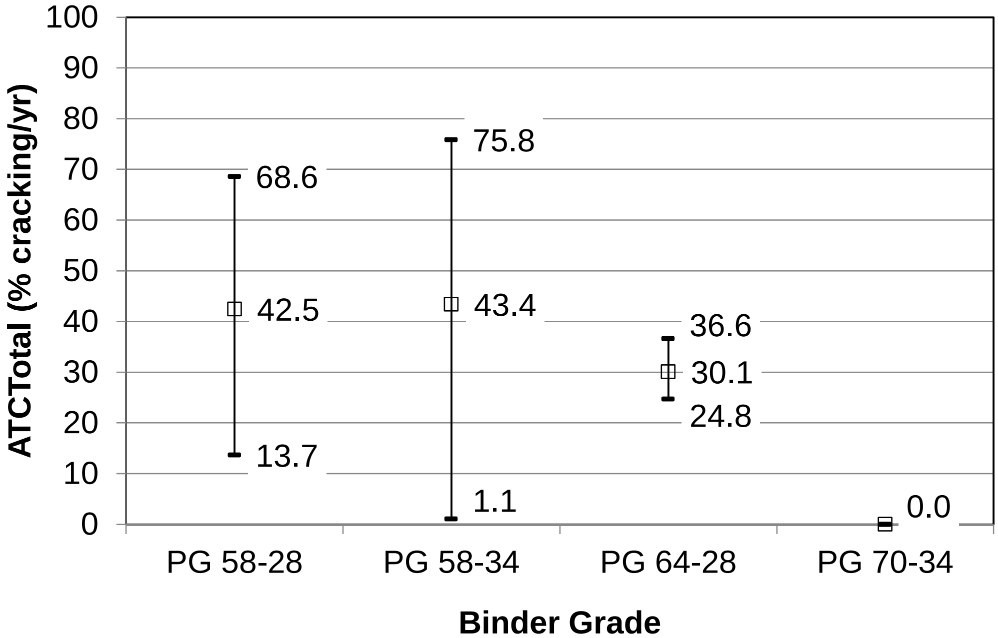

Figure 5.3: Effect of Asphalt Binder Grade on the Average Total Transverse Cracking Amount (ATCTotal)

5.2.2.2 PG Spread

The PG spread of the binder is defined as the total spread between the high and low performance grade temperatures for a binder. For example, the PG spread for PG 58-28 binder would be 86 (58 + 28). Figure 5.4 and Figure 5.5 exhibit similar trends to the PG grade plots. As the spread between the high temperature and low temperature of a binder increases, the transverse cracking performance of pavement deteriorates. However, Figure 5.6 showing the average transverse cracking amount does not exhibit this trend. It appears that the average transverse cracking amount is relatively independent of PG spread. This demonstrates that while the total cracking for a section with smaller PG spread would be higher, there may not be significant difference between the average annual cracking between sections with small or large PG spread. Trunk Highway 212 was analyzed as a separate data set. In the future, it would be beneficial to survey additional sections with PG 70-34 binder and/or SMA mix types to determine if the findings presented here in context of TH 212 are applicable to other highways.

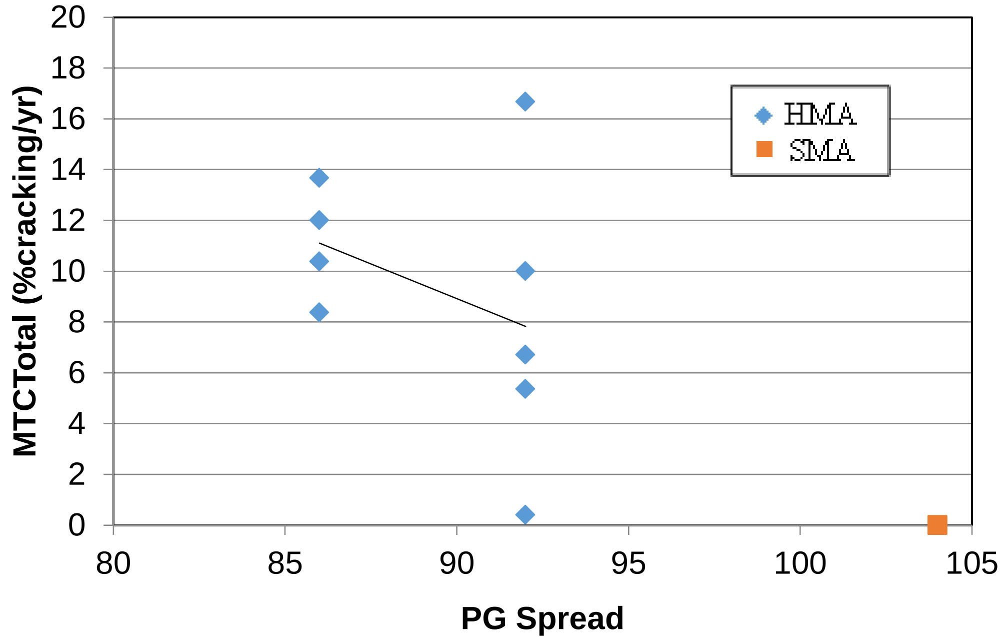

Figure 5.4: Effect of Performance Grade Spread of Asphalt Binder on the Maximum Total Transverse Cracking Amount (MTCTotal)

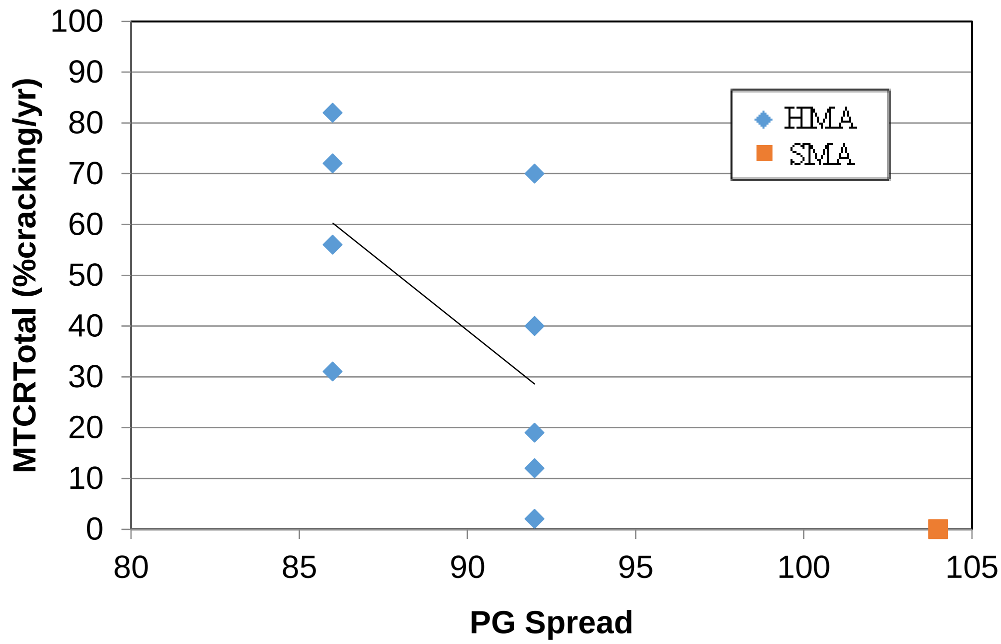

Figure 5.5: Effect of Performance Grade Spread of Asphalt Binder on the Maximum Total Transverse Cracking Rate (MTCRTotal)

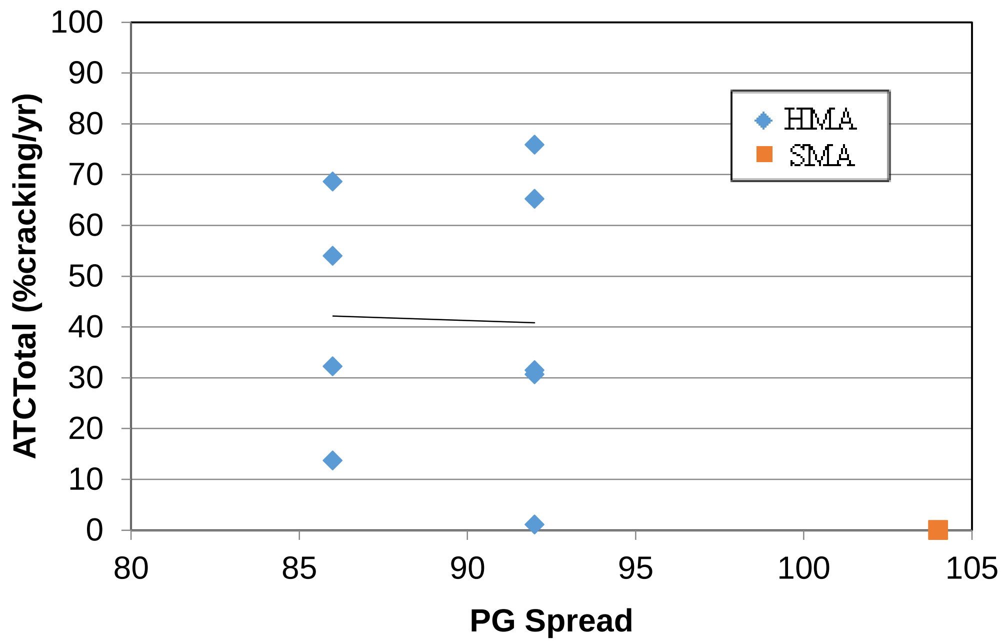

Figure 5.6: Effect of Performance Grade Spread of Asphalt Binder on the Average Total Transverse Cracking Amount (ATCTotal)

5.2.2.3 Total Asphalt Binder Content in the Mix

The plots analyzing the percent of asphalt content (Figure 5.7 to Figure 5.9) exhibit a general downward trend indicating superior transverse cracking performance for mixes with higher asphalt binder contents. This trend concurs with the conclusions from Task-1 of this study, claiming mixes with increased amounts of binder showed better transverse cracking resistance. Transverse cracking is the product of an asphalt pavement contracting under extreme low temperatures. If more asphalt is available to act as a medium for this "ductile straining" that occurs within the pavement system, it would seem reasonable that such a pavement would be more resistant to transverse cracking.

It should be noted that while the averaged trend shows improving cracking performance with increased asphalt binder content, there is significant scatter in the data, indicating that other factors may also be important and asphalt binder content alone cannot be used as an independent performance measure.

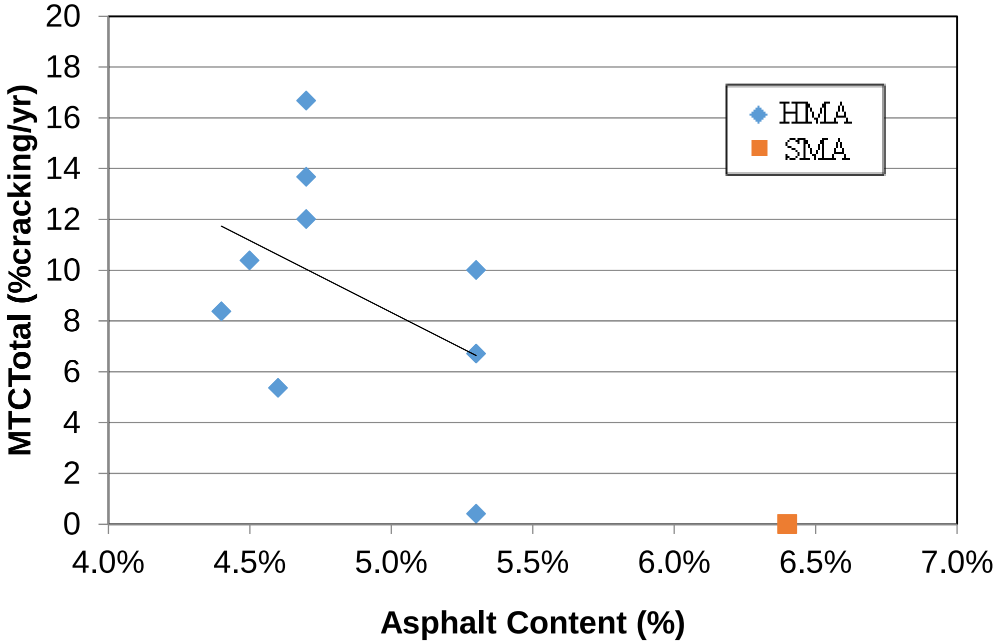

Figure 5.7: Effect of Percentage of Asphalt Content on the Maximum Total Transverse Cracking Amount (MTCTotal)

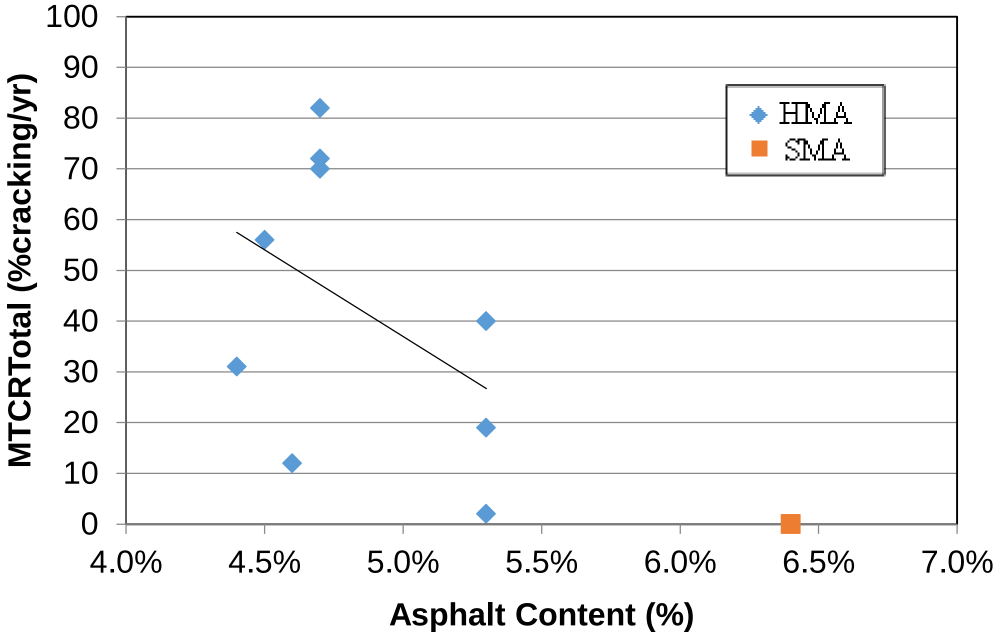

Figure 5.8: Effect of Percentage of Asphalt Content on the Maximum Total Transverse Cracking Rate (MTCRTotal)

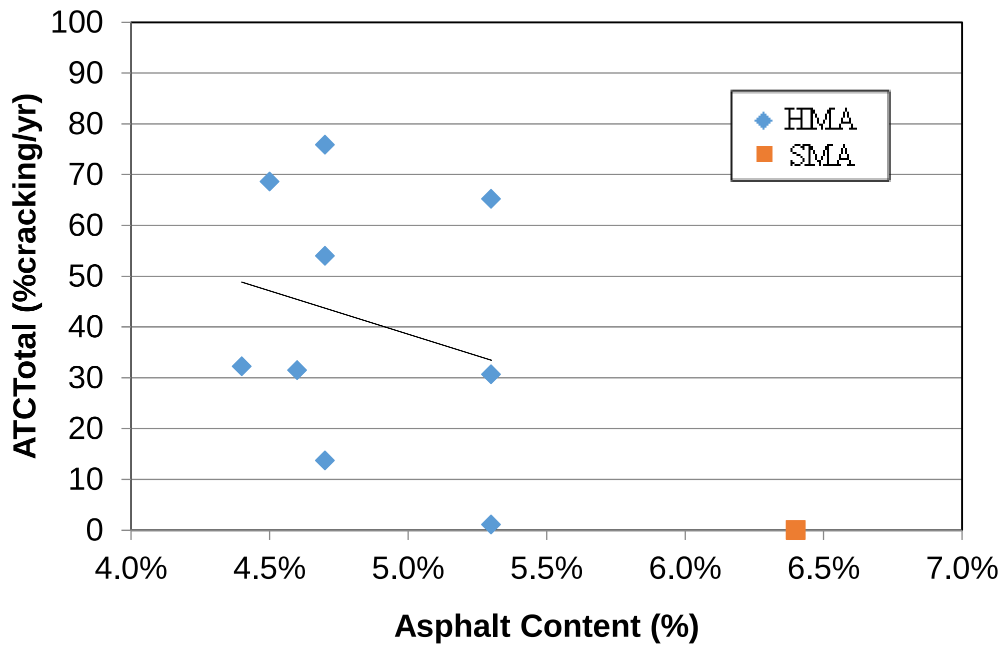

Figure 5.9: Effect of Percentage of Asphalt Content on the Average Total Transverse Cracking Amount (ATCTotal)

5.2.3 Effect of Amount of Recycled Asphalt Content on Transverse Cracking Performance

The impact of recycled asphalt content on transverse cracking is a multifaceted issue. While Figure 5.10 to Figure 5.12 show a slight downward trend as the amount of recycled asphalt binder increases, there are two sections with almost zero transverse cracking and less than 20% recycled binder. These two values would appear to follow the common philosophy that virgin binder results in a better performing pavement. It is difficult to draw any consistent conclusions since the amount of recycled asphalt binder is tied with many other variables such as: type and age of recycled asphalt pavement (RAP), type and amount of recycled asphalt shingles (RAS) and original grade of binder in recycled products. In this instance the scatter in the data seems to agree with presence of other variables. This once again supports the need for using laboratory testing based performance measure, such as DCT fracture energy, as opposed to using a mix design parameter as a performance control parameter. The fracture energies of the asphalt mixes studied herein will be determined through DCT testing of field sampled materials (Task-2B). The results will be analyzed as part of the Task-3B of this project.

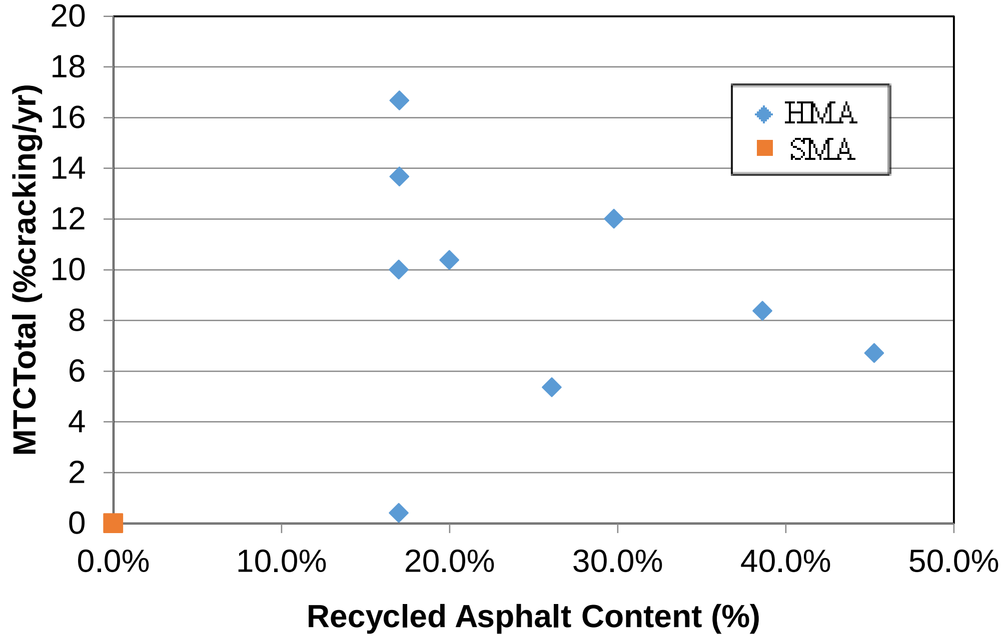

Figure 5.10: Effect of Recycled Asphalt Content on the Maximum Total Transverse Cracking Amount (MTCTotal)

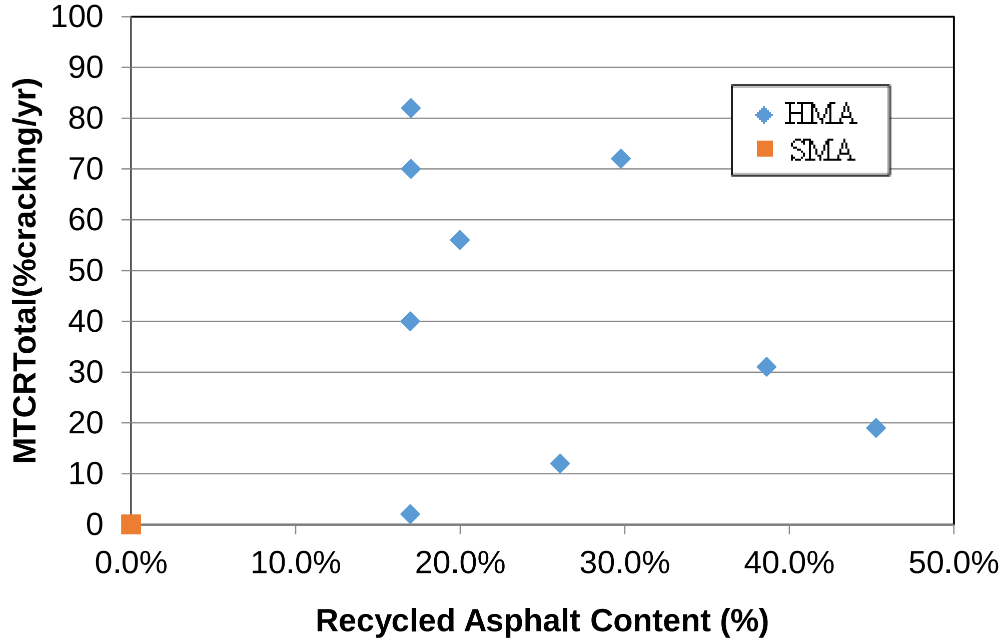

Figure 5.11: Effect of Recycled Asphalt Content on the Maximum Total Transverse Cracking Rate (MTCRTotal)

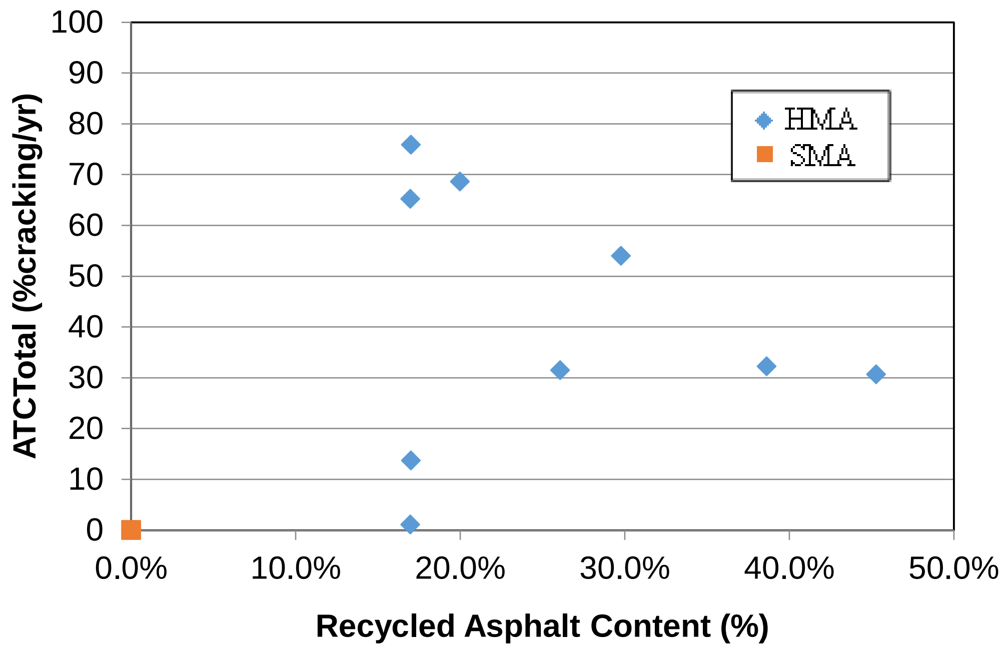

Figure 5.12: Effect of Recycled Asphalt Content on the Average Total Transverse Cracking Amount (ATCTotal)

5.2.4 Effect of Voids in Mineral Aggregate (VMA) on Transverse Cracking Performance

Figure 5.13 to Figure 5.15 show the comparison between transverse cracking measures and the voids in mineral aggregate (VMA) of each mix. Contrary to the recommendations from Task-1, the data in this portion of the study do not appear to have any type of trend in relation to VMA and transverse cracking. All the mixes in this study are three-quarter inch maximum aggregate size; therefore normalizing for recommended VMA values is not beneficial. It should be noted that except for two projects (TH 113 and TH 210) the remaining mixes used in this study were all designed and constructed using the older version of MnDOT 2360 specifications that required a minimum VMA amount. Both of the newer designs that utilized adjusted asphalt film thickness (AFT) specifications had significantly lower VMA amounts.

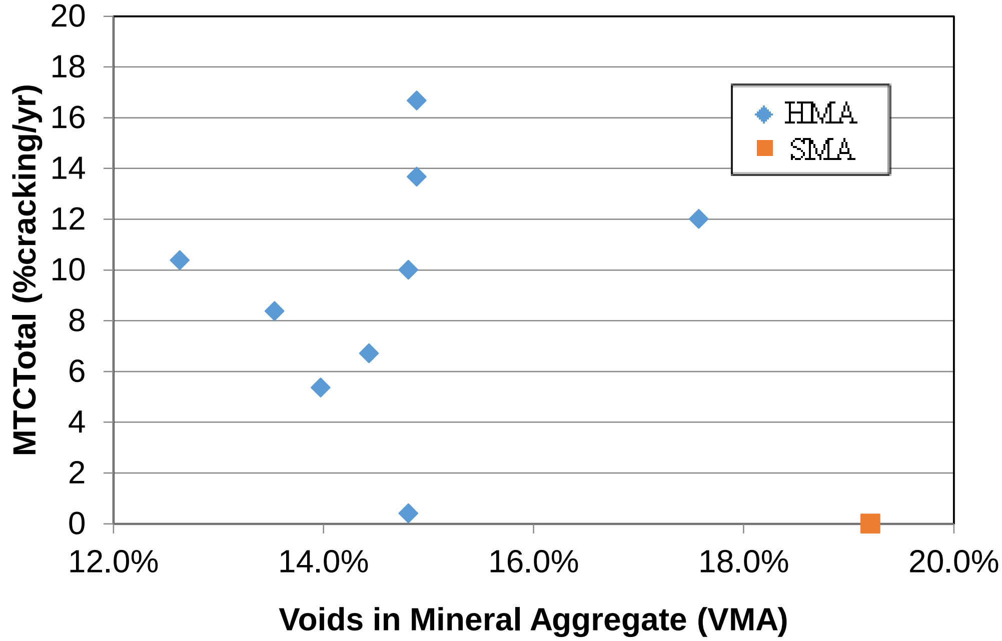

Figure 5.13: Effect of Voids in Mineral Aggregate (VMA) on the Maximum Total Transverse Cracking Amount (MTCTotal)

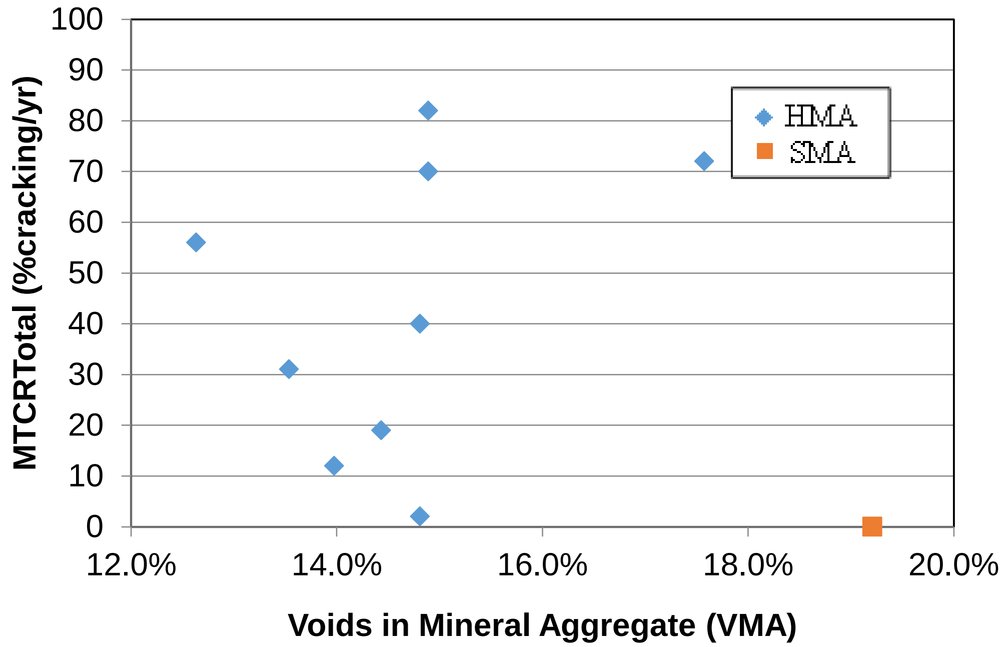

Figure 5.14: Effect of Voids in Mineral Aggregate (VMA) on the Maximum Total Transverse Cracking Rate (MTCRTotal)

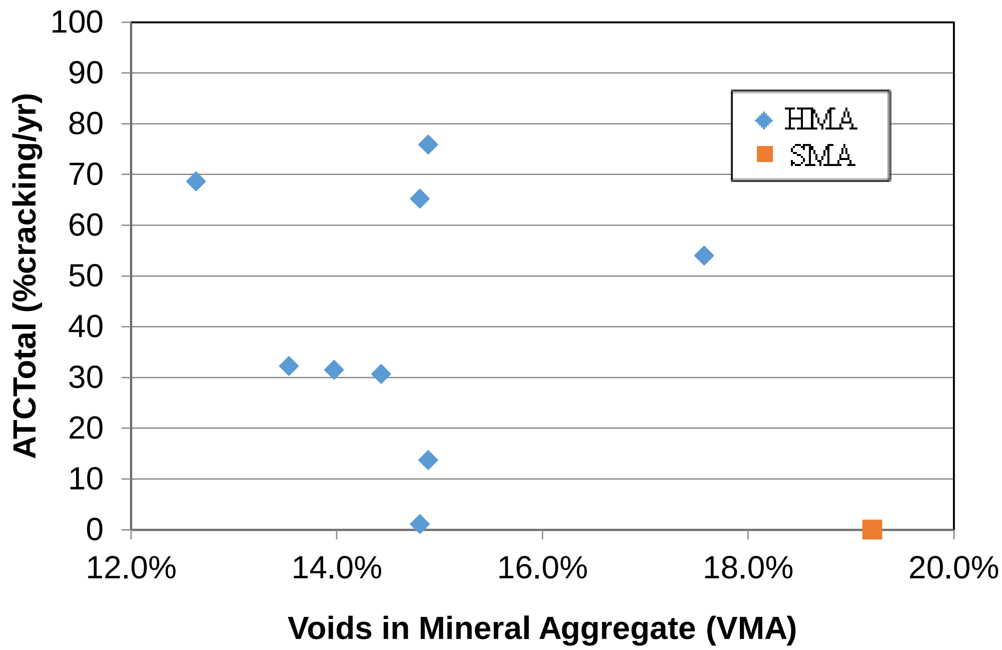

Figure 5.15: Effect of Voids in Mineral Aggregate (VMA) on the Average Total Transverse Cracking Amount (ATCTotal)

5.2.5 Effect of Voids Filled with Asphalt (VFA) on Transverse Cracking Performance

Figure 5.16 to Figure 5.18 show trends that result from the analysis of the voids filled with asphalt (VFA) of each mix. The data in this portion of the study does not appear to have any type of trend in relation to VFA and transverse cracking. While all the mixes in this study do not have the same design traffic level (basis for Superpave VFA recommendations), all of the mixes meet the suggested VFA range for the corresponding traffic level.

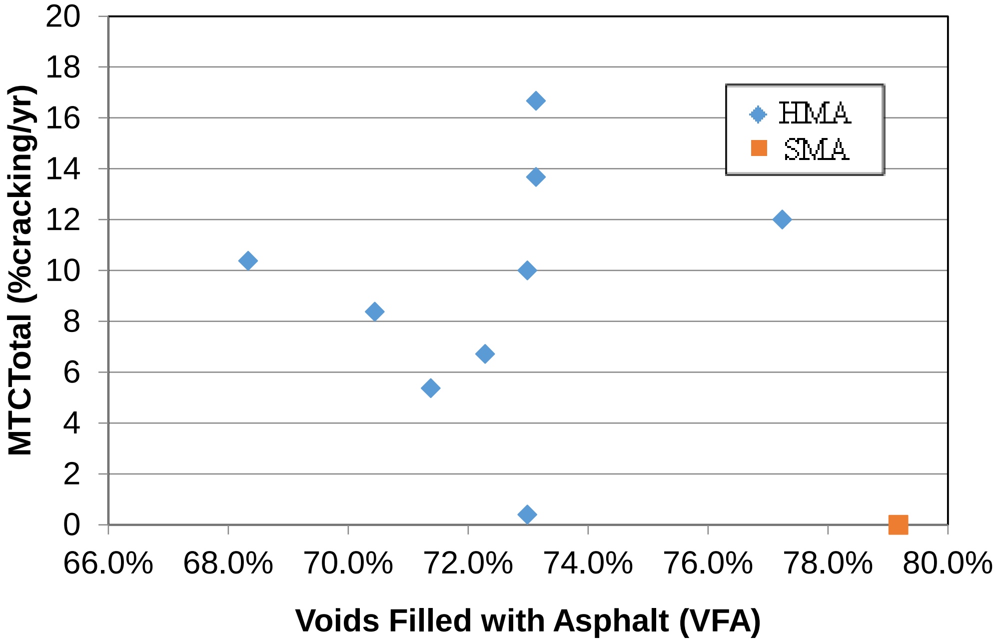

Figure 5.16: Effect of Voids Filled with Asphalt (VFA) on the Maximum Total Transverse Cracking Amount (MTCTotal)

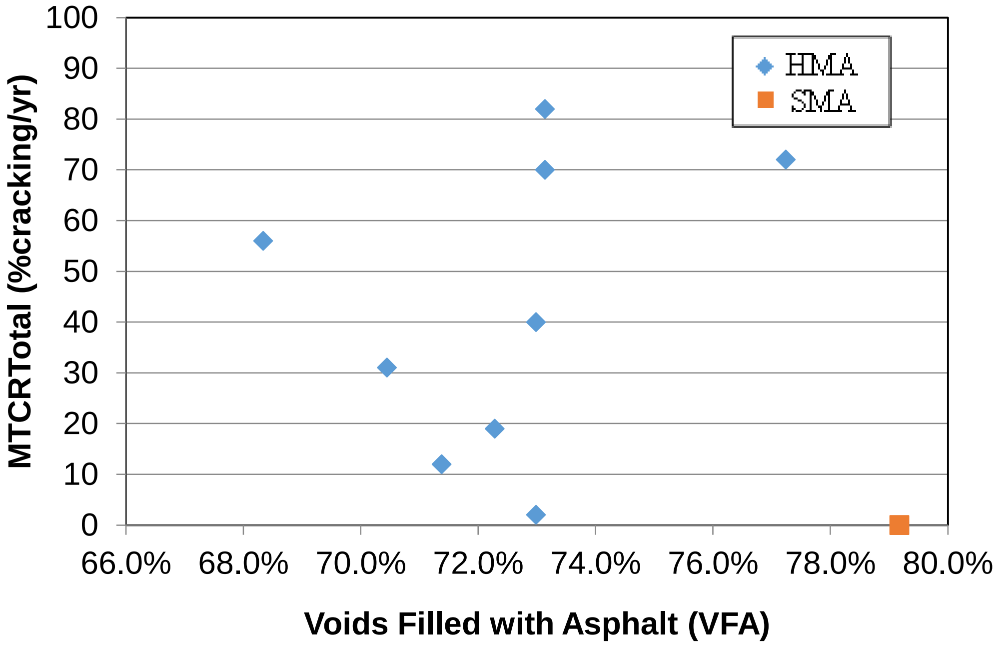

Figure 5.17: Effect of Voids Filled with Asphalt (VFA) on the Maximum Total Transverse Cracking Rate (MTCRTotal)

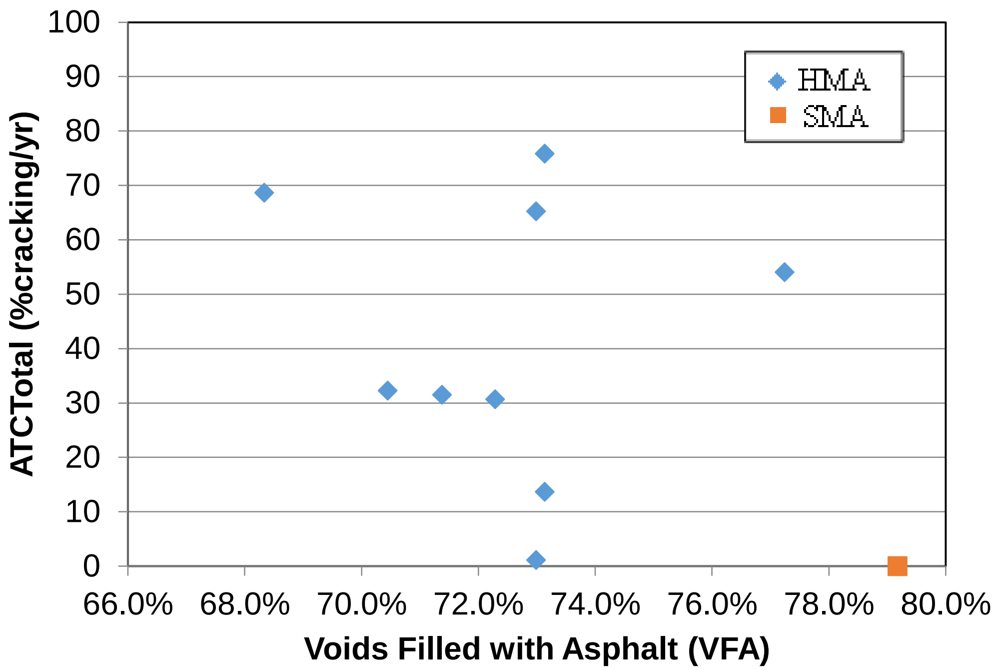

Figure 5.18: Effect of Voids Filled with Asphalt (VFA) on the Average Total Transverse Cracking Amount (ATCTotal)

5.2.6 Effect of Adjusted Asphalt Film Thickness (AFT) on Transverse Cracking Performance

The adjusted asphalt film thickness (AFT) for various mixes are plotted against the transverse cracking performance measures in Figure 5.19 thru Figure 5.21. For the mixes designed and produced using the older MnDOT 2360 specifications the adjusted AFT values were calculate using the information from MDRs and the mix test summary sheets (TSS).

The plots indicate a general trend of slightly deteriorating transverse cracking performance with increasing values of adjusted AFT. As with other parameters the data is still prone to significant scatter and the trend should not be used for the purposes of drawing conclusions. When the adjusted AFT is plotted against the maximum transverse cracking amount that is normalized for the varying asphalt layer thicknesses (c.f. Figure 5.22), the slightly increasing trend is less pronounced and the data indicates minimal effect of adjusted AFT on the cracking performance.

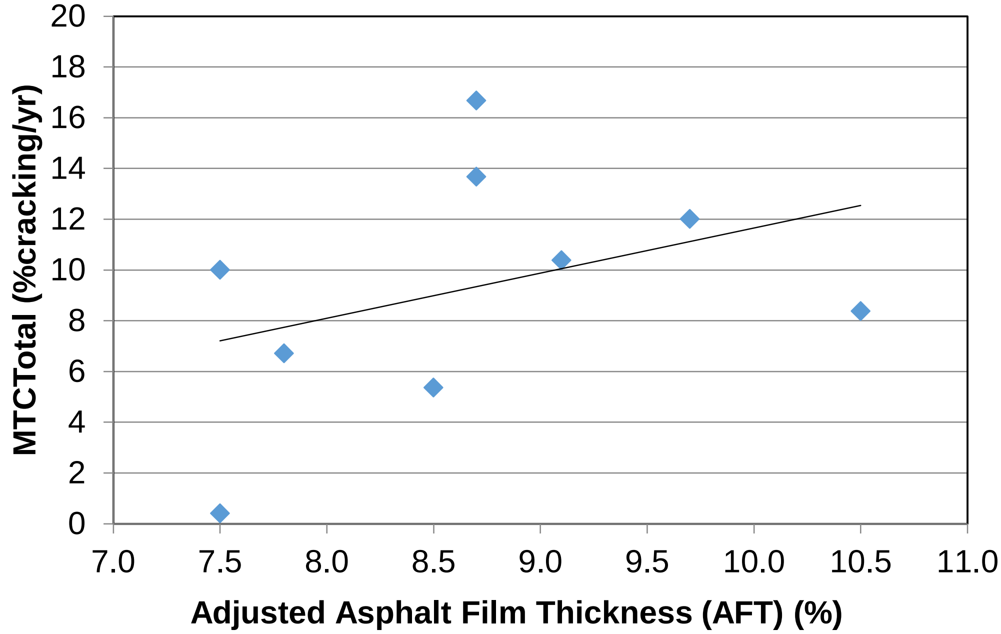

Figure 5.19: Effect of Adjusted Asphalt Film Thickness (AFT) on the Maximum Total Transverse Cracking Amount (MTCTotal)

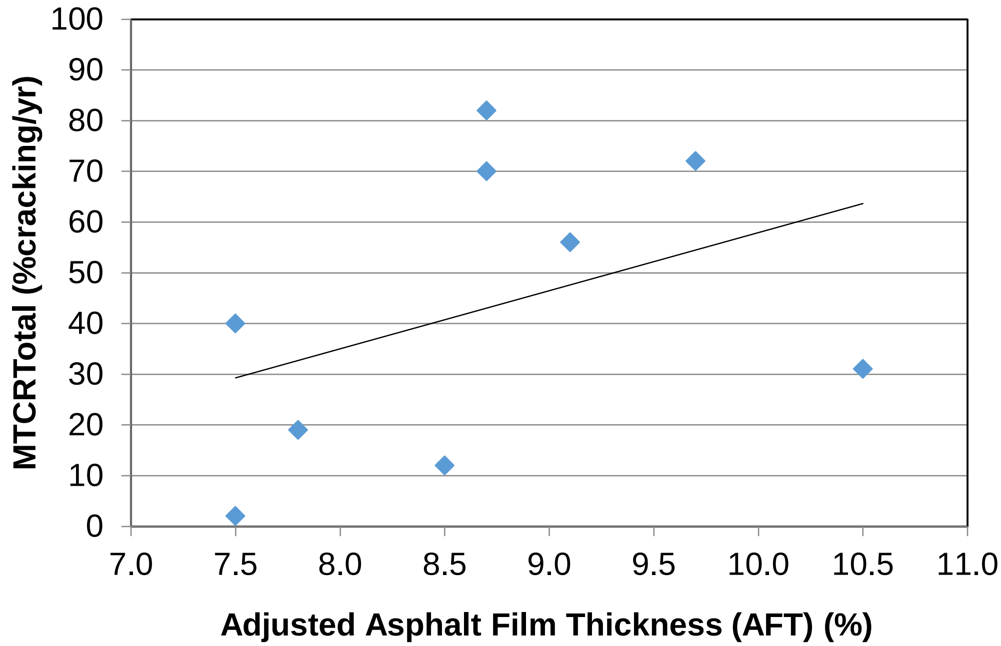

Figure 5.20: Effect of Adjusted Asphalt Film Thickness (AFT) on the Maximum Total Transverse Cracking Rate (MTCRTotal)

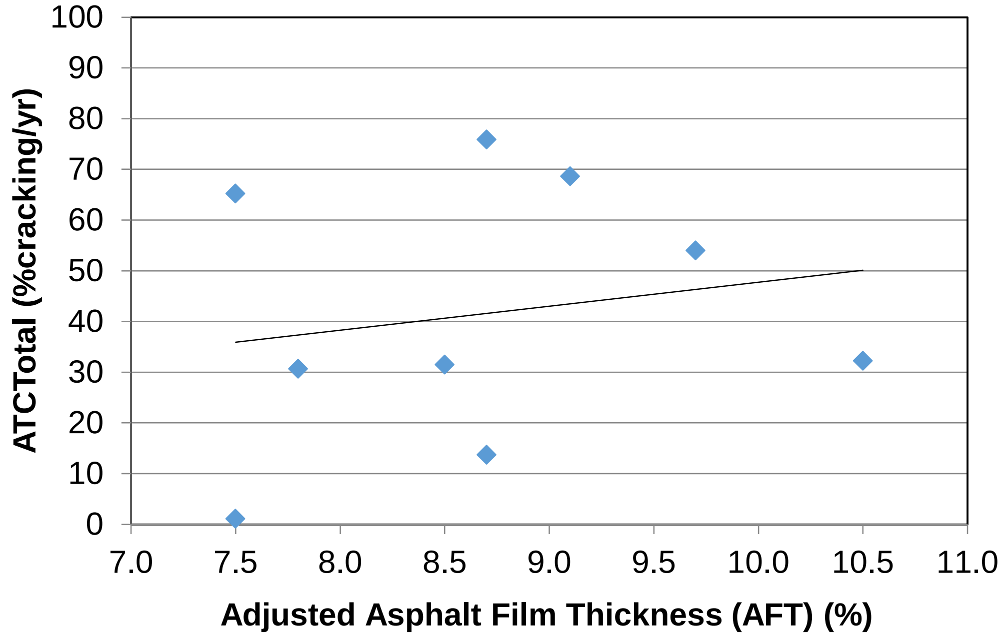

Figure 5.21: Effect of Adjusted Asphalt Film Thickness (AFT) on the Average Total Transverse Cracking Amount (ATCTotal)

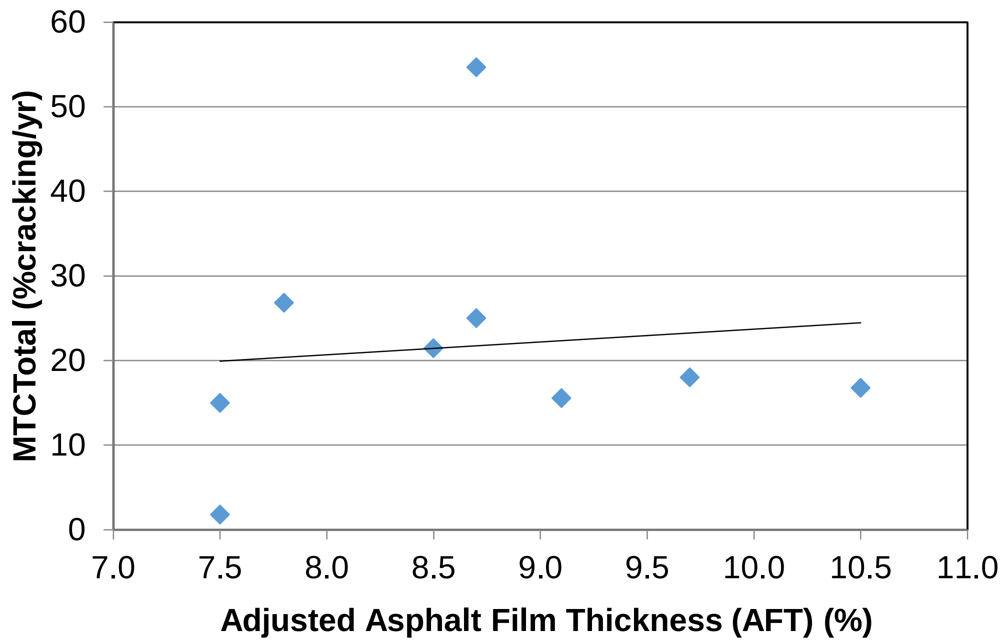

Figure 5.22: Effect of Adjusted Asphalt Film Thickness (AFT) on the Maximum Total Transverse Cracking Amount (MTCTotal) Normalized for the Asphalt Layer Thickness

5.3 Comparison of Pavement Construction Type with Transverse Cracking Performance (Task-3A)

5.3.1 Introduction

During the analysis of mix design parameters, it became apparent that construction types may have an impact on transverse cracking performance. In this study there was several construction methods used with various asphalt layer depths. It is not practical to attempt and relate each variation to cracking performance. For analysis purposes, three primary construction types were identified: overlays, reclaimed asphalt and new construction. Any section with an asphalt wear course on an existing roadway is herein referred to as an “overlay”. Sections with overlay on a reclaimed asphalt layer will be considered a “reclaim”. The only section that featured new construction was Trunk Highway 212. Historical records show no cracking on this section. This data point is incorporated in the following plots, but the lack of sufficient data for new construction cracking is recognized.

5.3.2 Effect of Mix Parameters on Cracking Performance for Different Construction Types

Generally, mix design parameters did not show a strong trend relating construction type and transverse cracking performance. Figure 5.23 and Figure 5.24 show two parameters that did exhibit a potential relationship between construction type and field performance. In Figure 5.23 as PG spread increases from 86 to 92, transverse cracking in reclaim sections shows a significant improvement while overlays exhibit less improvement. Figure 5.24 shows the same trend as asphalt content increases. The preliminary trend in this instance is that it appears to be advantageous to use higher asphalt content or larger PG spread in reclaim sections as opposed to overlays. This trend should continue to be monitored in future studies.

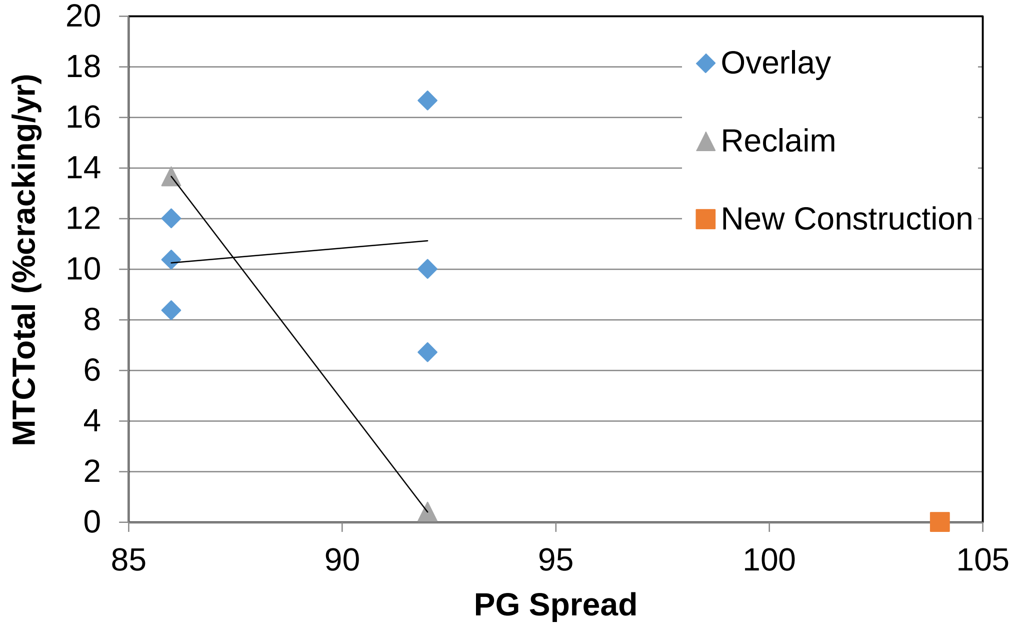

Figure 5.23: Effect of Performance Grade Spread on the Maximum Total Transverse Cracking Amount (MTCTotal) Categorized by Construction Type

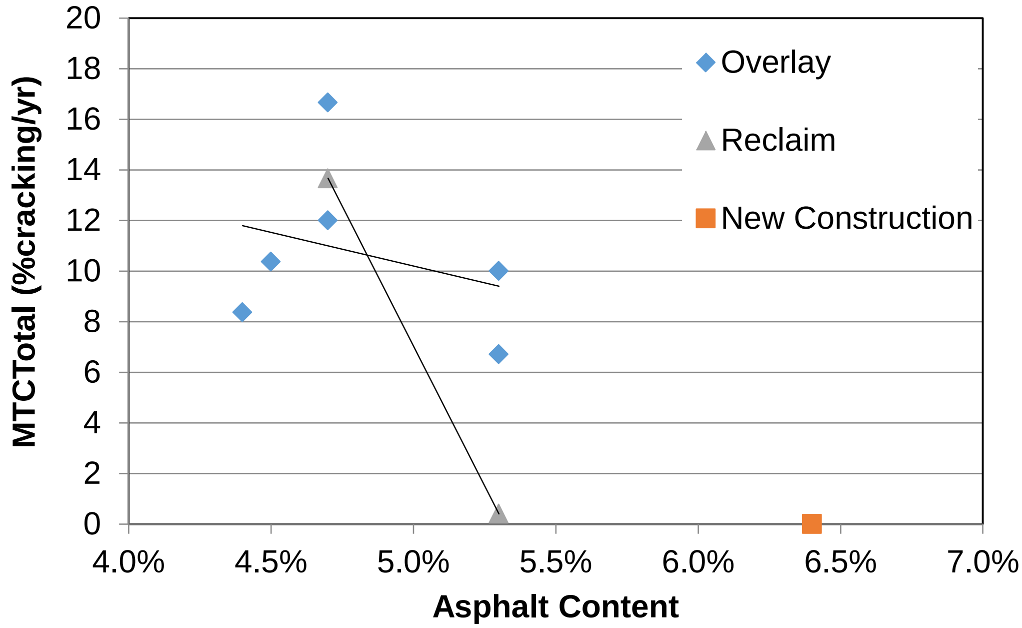

Figure 5.24: Effect of Percentage of Asphalt Content on the Maximum Total Transverse Cracking Amount (MTCTotal) Categorized by Construction Type

5.3.3 Effect of Pavement Section Type on Cracking Performance

Observing the data in Figure 5.25 thru Figure 5.27, the construction methods are arranged in the following order: overlay, reclaim and new construction. In these plots, there is a general trend of decreasing transverse cracking as the plot progresses left to right. Figure 5.26 is best understood viewed with Figure 5.27. While asphalt reclamation projects appear to result in greater rates of transverse cracking (Figure 5.26), the average amount of transverse cracking present on a yearly basis for the reclaim sections is significantly lower than overlay projects (Figure 5.27). In other words, reclaim sections often see a significant increase in the amount of cracking over certain year during their life, but this usually happens later in the service life as opposed to overlays where the high cracking rate occurs early in the service life. For example, comparisons between the overlay and reclaim sections from TH 1 (Figure 5.28) show that the overlay section experienced 70% cracking in first year of service, whereas, the reclaim section did not experience any significant cracking until year 6.

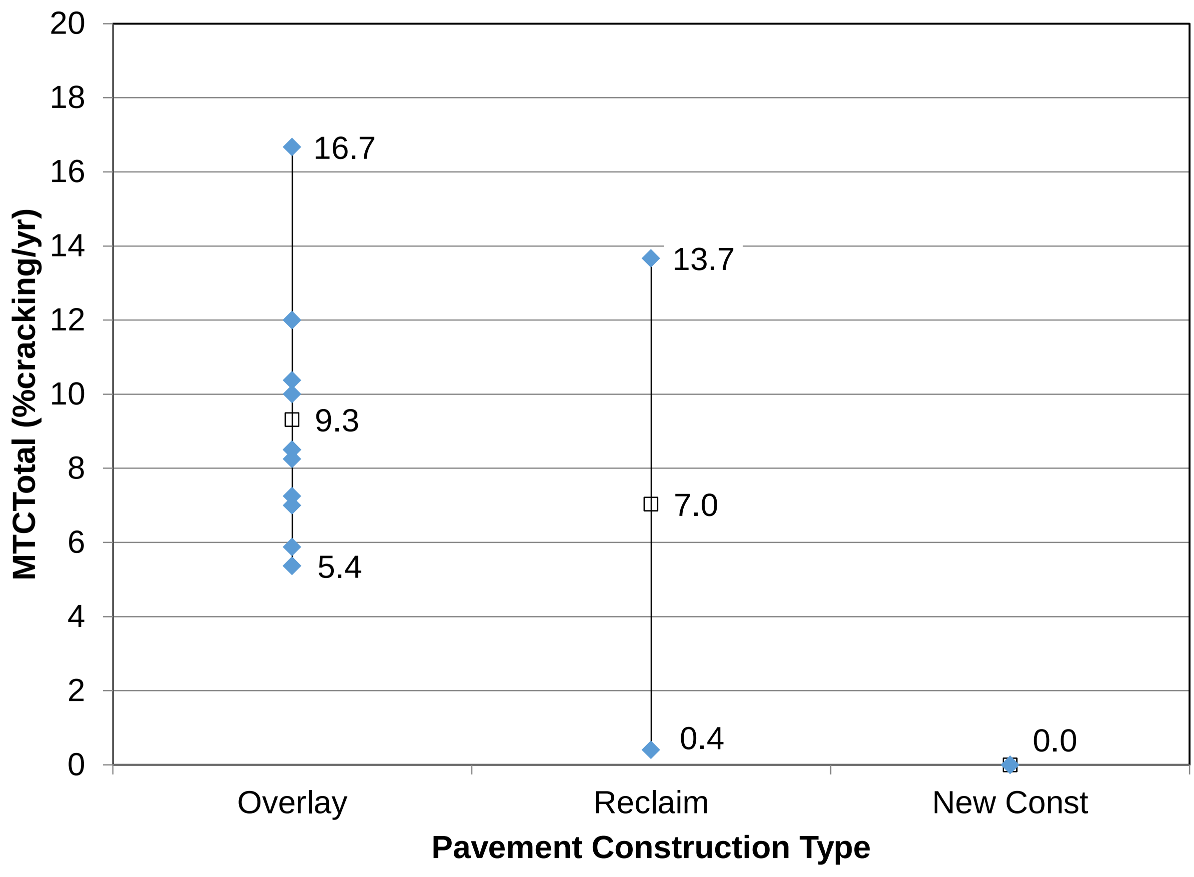

Figure 5.25: Comparison of Maximum Total Transverse Cracking Amounts (MTCTotal) between Construction Types

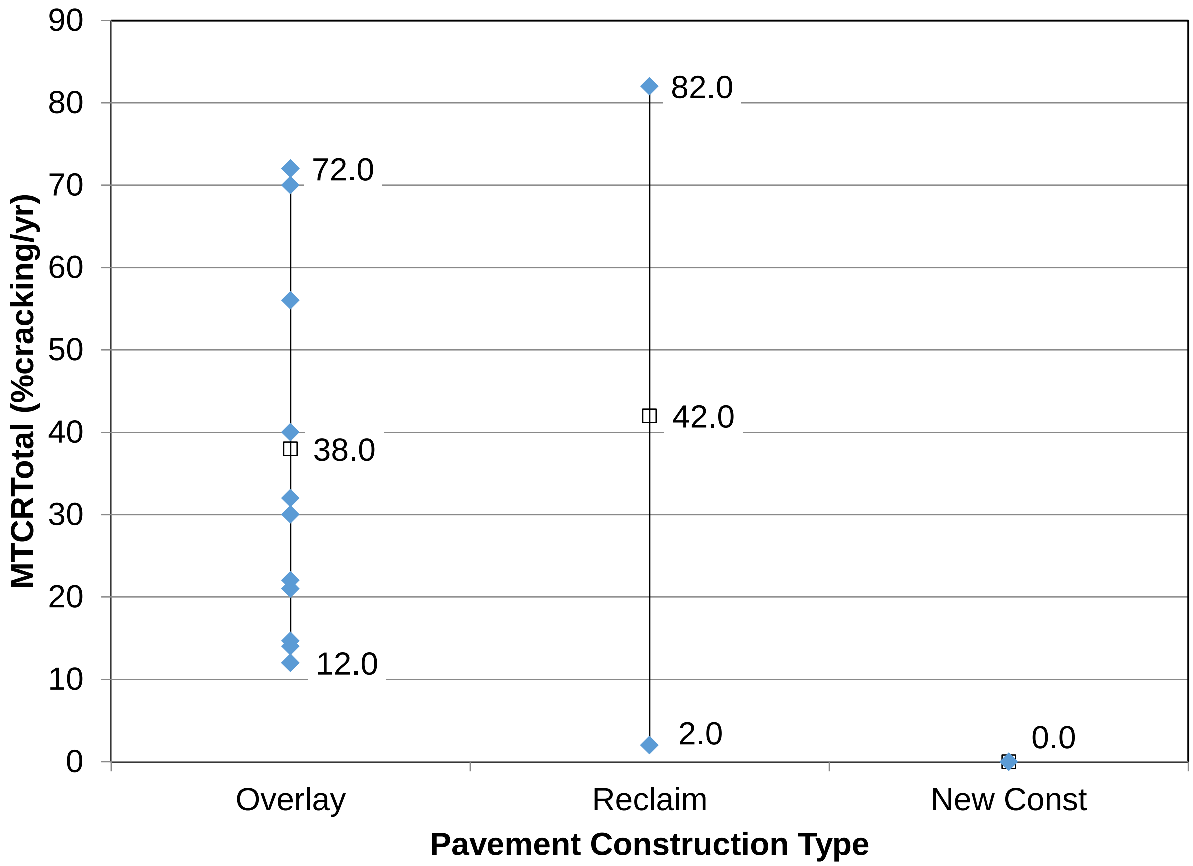

Figure 5.26: Comparison of Maximum Total Transverse Cracking Rates (MTCRTotal) between Construction Types

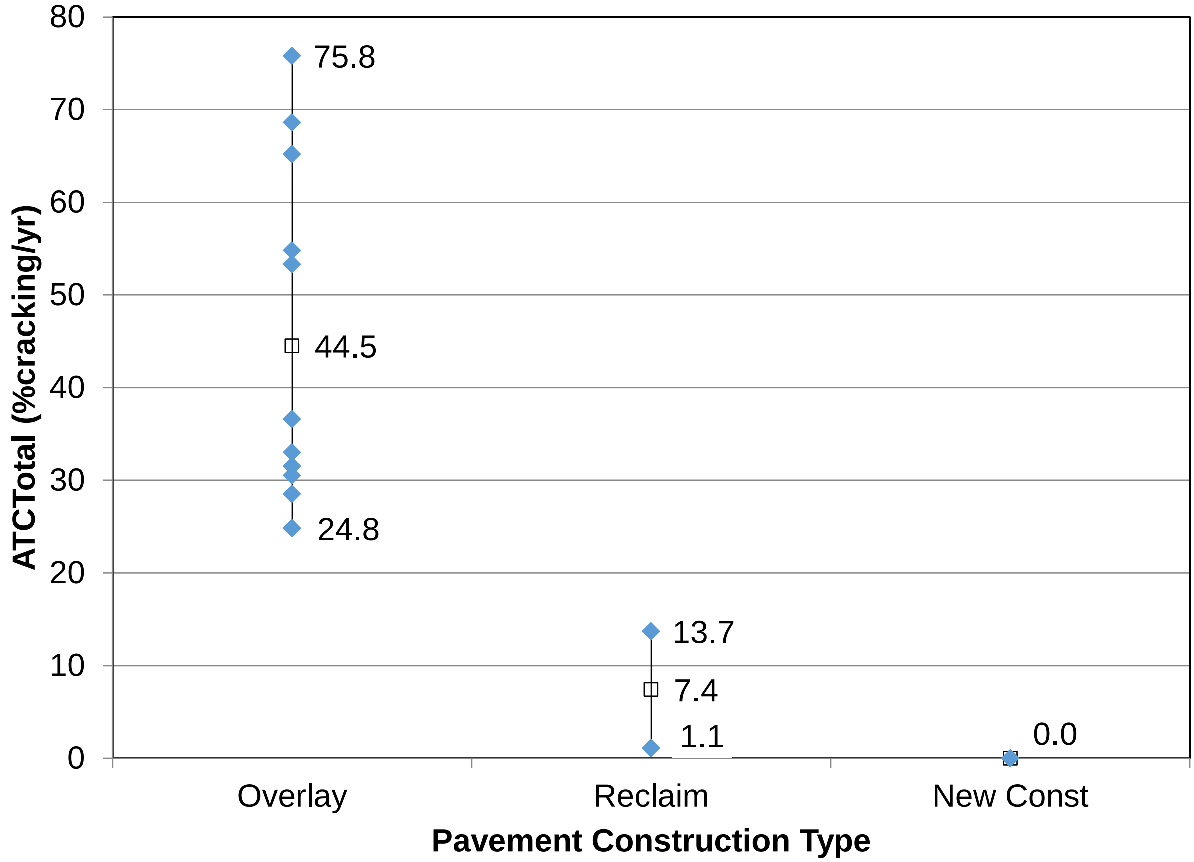

Figure 5.27: Comparison of Average Total Transverse Cracking Amounts (ATCTotal) between Construction Types

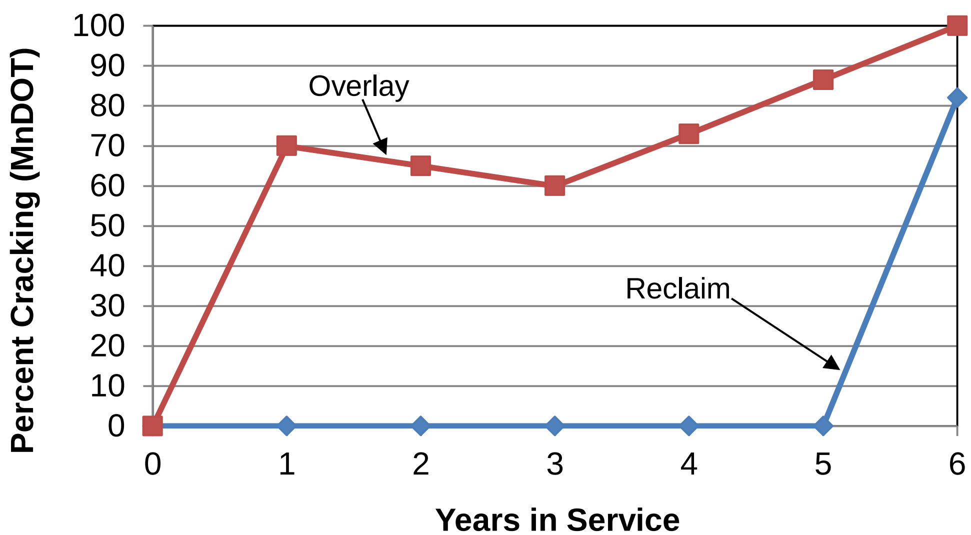

Figure 5.28: Cracking Performance of TH 1 (SP 8821-103)

In addition to Figure 5.27, Figure 5.29 has been provided to reinforce the point that the reclaim sections from this project exhibit a greater resistance to transverse cracking. Figure 5.29 has been normalized against the asphalt layer thickness. As explained in Section 5.1.2, normalization of asphalt layer thickness is conducted by multiplying the transverse cracking amounts with the total asphalt layer thickness. In a general sense reclaim projects tend to feature a thicker asphalt layer than overlays. After normalizing for the asphalt layer thickness, reclaim sections still exhibit a superior transverse cracking resistance to overlay sections. Thus, even when the added cost of thicker asphalt layer is accounted for in the analysis the cracking performance is still superior with reclaim sections. It should be noted again that this task features a small amount of sections and these trends are not identified with a high level of confidence. These are initial observations and will continue to be observed as more data is acquired.

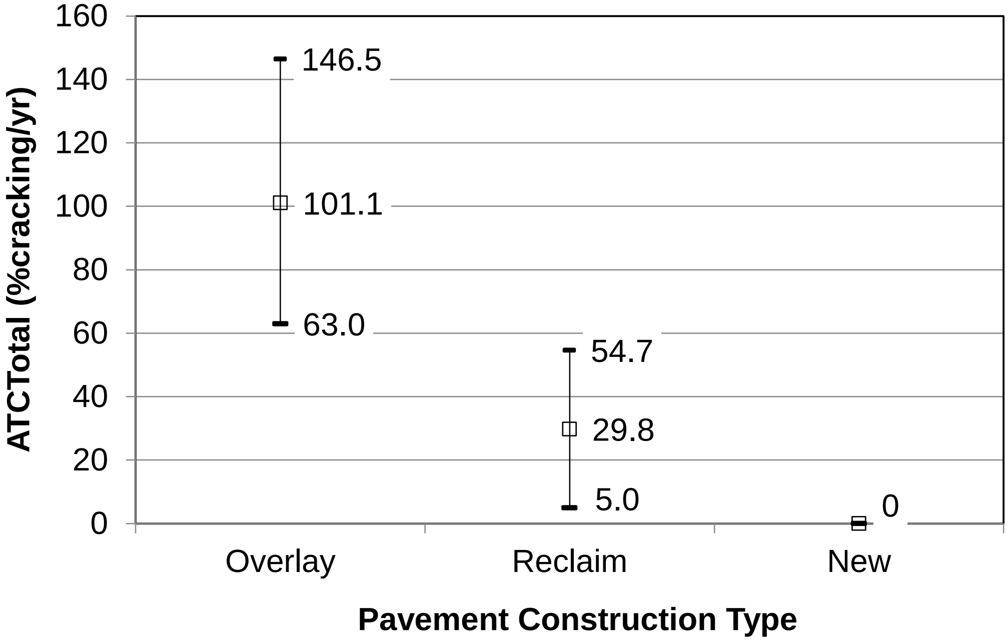

Figure 5.29: Comparison of Average Total Transverse Cracking Amounts (ATCTotal) between Construction Types Normalized against Asphalt Layer Thickness

5.4 Comparison of Mix Design Parameters with Fracture Energy of Pavement Sections (Task-3B)

5.4.1 Introduction

The asphalt mixture parameters were determined from the Mix Design Records (MDR) for each mix type obtained from the MnDOT Bituminous Engineering Office. The MDRs for each of the mixes are attached in the Appendix-I. Sections 5.4.2 through 5.4.6 of this report detail the analysis of the primary mix design parameters listed in Section 5.1.2 with DCT fracture energy. In all of the following analysis procedures, Trunk Highway 212 is relegated to a separate series from the pooled section cracking data. This is due to the following factors: (1) the site was not visually surveyed, (2) a stone-matrix asphalt (SMA) mix is used; and, (3) the binder grade (PG 70-34) is the only one of its type in this study. It is shown in the analysis for completeness, but not considered influential to any of the trends mentioned in the following sections.

The mix parameters of interest for each section in the study can be found in Table 5.1. Given the limitations of this study, the various roadway sections provide a fair amount of variability for each parameter. This variability provides a sufficient amount of data for determining any preliminary relationships between the individual values and fracture energy. Refer to the following subsections for detailed discussions on the effect of each parameter.

For any plots featuring a best fit regression line, it should be noted that the intention of this is not to show linear uniformity. The placement of this linear regression line is to simply show the approximate trend for the data being presented. In the data that follows, the blue markers represent all sections except for Trunk Highway 212 which is represented with a red marker. This is the typical condition unless explicitly defined in an alternative manner. In several instances multiple lanes (passing, driving etc.) were surveyed for same pavement section. Throughout this report the cracking performance for such sections are presented as average values for all lanes.

A notable challenge with this study is the influence of binder aging. Each individual section will feature a variable (and unknown) amount of aging in the corresponding binder. This is due to each binder aging at differing rates. The sections will inevitably see different climatic conditions. Fracture energy will be influenced by the age of the binder, with brittle binders providing a lower fracture energy than a ductile binder. Therefore, the age of binder in field cores can have an unpredictable effect on fracture energy performance. Simply normalizing fracture energy for years in service does not alleviate this issue, as total service life does not necessarily correlate to binder age. This factor can lead to additional uncertainty when comparing fracture energy between sections.

It should also be noted that the mixes in this study were developed using different material specifications. Some mixes were products of the VMA based specification, while others were established using the AFT based specification. These specifications have inherent differences. Some of these differences will be covered within this report (AFT, VMA). However, other unknown contributing factors from the use of these specifications may influence the results presented herein.

Table 5.1: Summary of asphalt mixture parameters by section

Section	SP #	RP / Landmark	PG Grade	PG Spread	Asphalt Content	Recycled Asphalt Content	Voids in Mineral Aggregate (VMA)	Voids Filled with Asphalt (VFA)
TH 1	8821-103	RP 235	58-34	92	4.7%	17.0%	14.9%	73.1%
TH 1	8821-103	RP 230	58-28	86	4.7%	17.0%	14.9%	73.1%
TH 2	1102-59	RP 157	58-34	92	4.6%	26.1%	14.0%	71.4%
TH 6	3107-42	RP 118	58-34	92	5.3%	17.0%	14.8%	73.0%
TH 6	3107-42	RP 123	58-34	92	5.3%	17.0%	14.8%	73.0%
TH 10	0502-95	RP 159	64-28	92	5.3%	45.3%	14.4%	72.3%
TH 10	0502-95	RP 161	64-28	92	5.3%	45.3%	14.4%	72.3%
I-35	0283-26	N/A	64-28	92	5.0%	34.0%	15.1%	73.5%
TH 53	8821-177	169 to Ely	58-28	86	4.7%	29.8%	17.6%	77.2%
TH 113	4407-12	RP 10	58-28	86	4.5%	20.0%	12.6%	68.3%
TH 113	5413-10	RP 5	58-34	92	4.5%	20.0%	12.6%	68.3%
TH 210	1805-72	RP 118	58-28	86	4.4%	38.6%	13.5%	70.4%
TH 212	1017-12	RP 147	70-34	104	6.4%	0.0%	19.2%	79.2%

5.4.2 Effects of Asphalt Binder on Fracture Energy

5.4.2.1 PG Grade

Figure 5.30 presents the plot of PG grade against fracture energy. This plot incorporates a box-and-whisker design, where the average is the "box" with the maximum and minimum values as the "whiskers". The average values are for all pavement sections that were constructed using same grade of the virgin binder in the mix. The plots are generated in the order of increasing average fracture energies with the PG grades. From these plots a loose trend of increasing fracture energy is observed as the binder grade goes in the order of PG 58-28, PG 64-28, PG 58-34 and PG 70-34. In comparison to the Task-3A report, PG 58-34 exhibited slightly higher transverse cracking amounts than that of PG 64-28 sections. However, the results are quite similar.

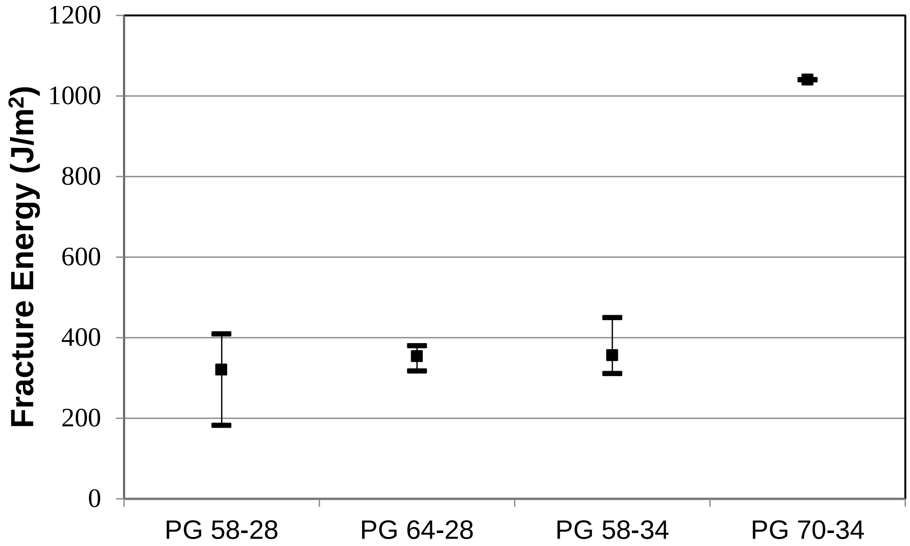

Figure 5.30: Effect of PG grade on fracture energy

5.4.2.2 PG Low Temperature (LT) Grade

Figure 5.31 presents the plot of PG low temperature (LT) grade against average fracture energies. As with Figure 5.30, this plot also incorporates a box-and-whisker design. The average values are for all pavement sections that were constructed using same PG LT grade. The plots are generated in the order of increasing fracture energy with the PG LT grades. TH 212 consisted of a PG 70-34 binder, the only representative of that grade in the study. Thus two data sets are provided for PG LT XX-34: one with TH 212 results and one without. From these plots a loose trend of increasing average fracture energy is observed as the low temperature binder grade goes in the order of PG XX-28, PG XX-34 (without TH 212) and PG XX-34 (all sections).

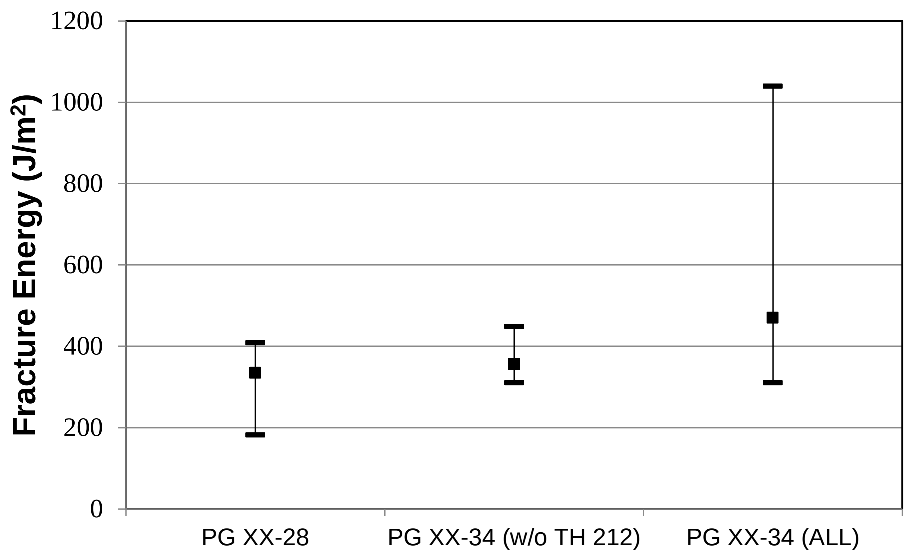

Figure 5.31: Effect of PG LT on fracture energy

5.4.2.3 PG Spread

The PG spread of the binder is defined as the total spread between the high and low performance grade temperatures for a binder. For example, the PG spread for PG 58-28 binder would be 86 (58 + 28). Figure 5.32 represents a similar trend to the PG grade plots. As the spread between the high temperature and low temperature of a binder increases, the average fracture energy of the study group increases accordingly. This appears to suggest that as the PG spread increases, the fracture energy also increases. Trunk Highway 212 was the only PG 70-34 binder in the study. Therefore, the spread of 104 only applies to one average fracture energy. The results from this mix are exceptional, with an average fracture energy of 1040 J/m^2. In the future, it would be beneficial to survey additional sections with PG 70-34 binder and/or SMA mix types to determine if the findings presented here in context of TH 212 are applicable to similar asphalt mixes.

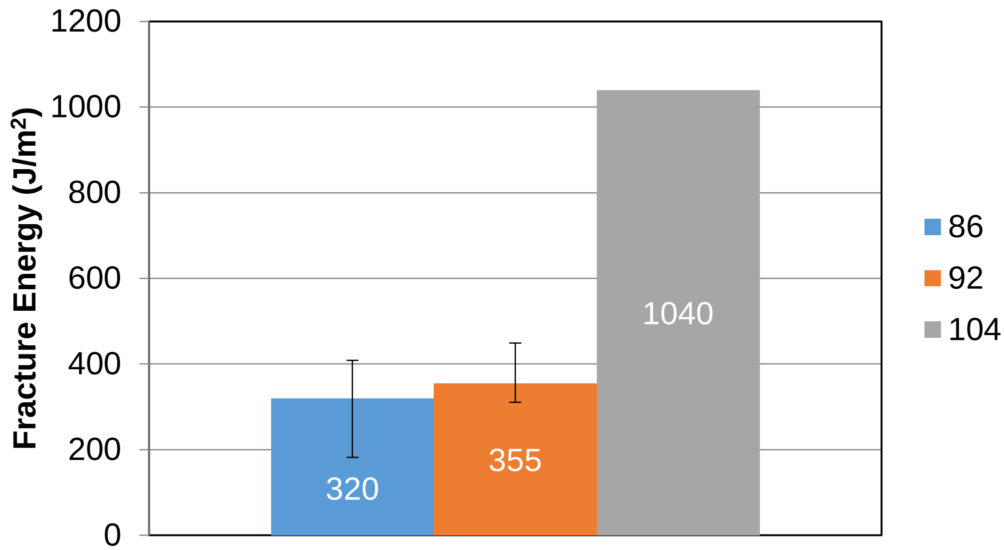

Figure 5.32: Effect of PG spread on fracture energy

5.4.2.4 Total Asphalt Binder Content in the Mix

The plots analyzing the percent of asphalt content (Figure 5.33 and Figure 5.34) exhibit a general upward trend indicating superior DCT testing performance for mixes with increasing asphalt binder contents and thereafter a slight reduction with further increase in binder amounts. Figure 5.34 is provided to facilitate the observation of the typical “cluster” of data, as TH 212 results make this observation difficult in Figure 5.33. The initial upward trend concurs with the conclusions from Task-1 and Task-2B of this study, claiming mixes with increased amounts of binder showed lower levels of transverse cracking, thus a higher fracture energy. Majority of transverse cracking results from thermally induced tensile stresses in asphalt pavement. If more asphalt is available to act as a medium for this “ductile straining” that occurs within the pavement system, it would seem reasonable that such a pavement would be more resistant to transverse cracking that occurs during DCT testing. However, for mixes with asphalt binder contents above this optimal range the fracture energy begins to drop as the mix starts to behave in exceedingly flexible manner and loses its capacity to carry tensile stresses.

The amount of scatter between data points appears to decrease as the amount of binder increases. This would seem reasonable as an asphalt mix with higher binder content could rely less on the aggregate structure and more on the elastic properties of the binder during DCT testing. This would result in more consistent results with mixes of higher binder content, as aggregate types and configurations can vary greatly between DCT specimens, even those of the same mix type.

It should be noted that while the averaged trend shows improving cracking performance with increased asphalt binder content until an optimal level is reached; there is significant scatter in the data, indicating that other factors may also be important and asphalt binder content alone cannot be used as an independent performance measure. In other words, these figures show that simply increasing asphalt binder content will not necessarily result in higher fracture energy. This reinforces the need for use of performance testing based specifications over those based entirely on volumetric controls.

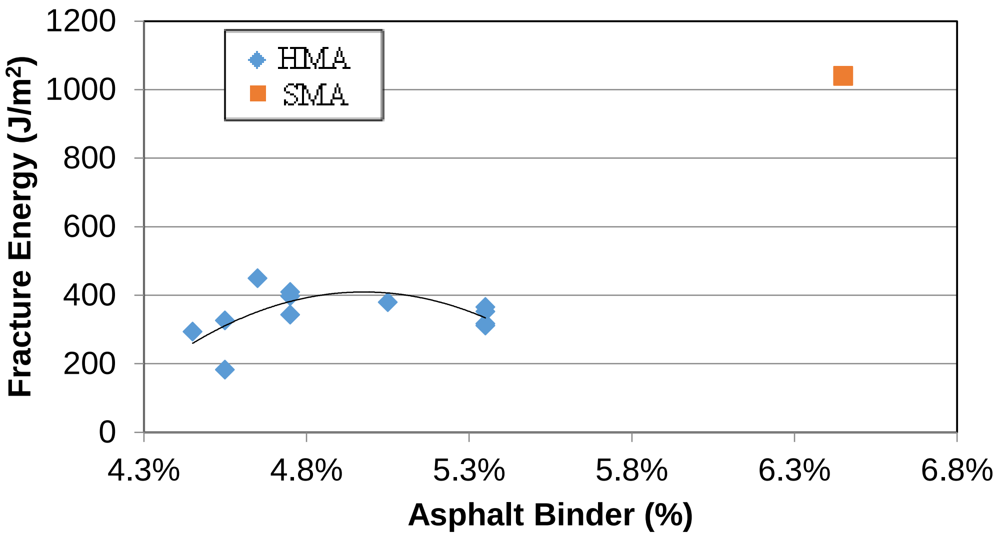

Figure 5.33: Effect of asphalt binder content (%) on fracture energy

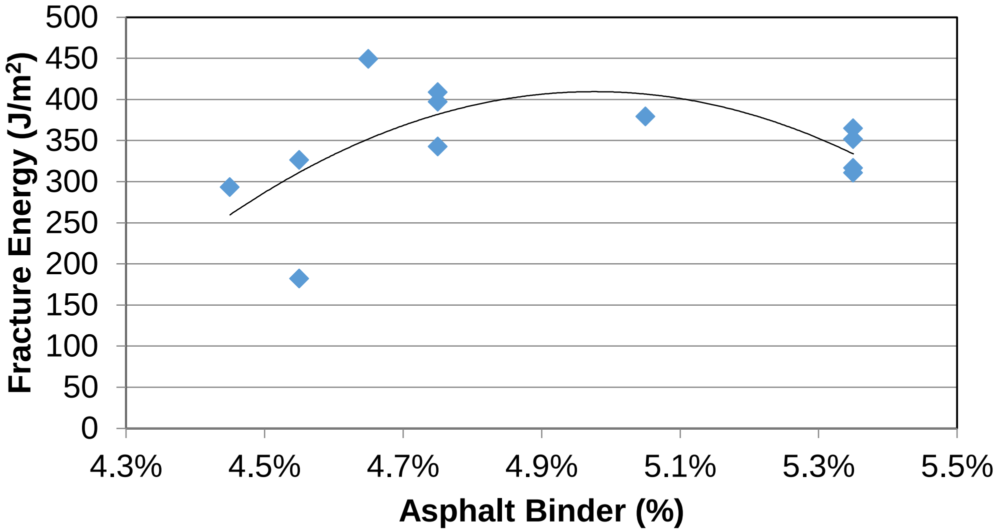

Figure 5.34: Effect of asphalt binder content (%) on fracture energy--excluding TH 212

5.4.3 Effect of Amount of Recycled Asphalt Content on Fracture Energy

The impact of recycled asphalt content on transverse cracking is a multifaceted issue. It is difficult to draw any consistent conclusions since the amount of recycled asphalt binder is tied with many other variables such as: type and age of recycled asphalt pavement (RAP), type and amount of recycled asphalt shingles (RAS) and original grade of binder in recycled products. Additionally, Figure 5.35 does not appear to suggest any strong relationship exists. In this instance the scatter in the data seems to agree with presence of other variables. This once again supports the need for using laboratory testing based performance measures, such as DCT fracture energy, as opposed to using a mix design parameter as a performance control parameter. The fracture energies of the asphalt mixes studied herein have been determined through DCT testing of field sampled materials, and their relationship to transverse cracking can be found in the Task-2B report.

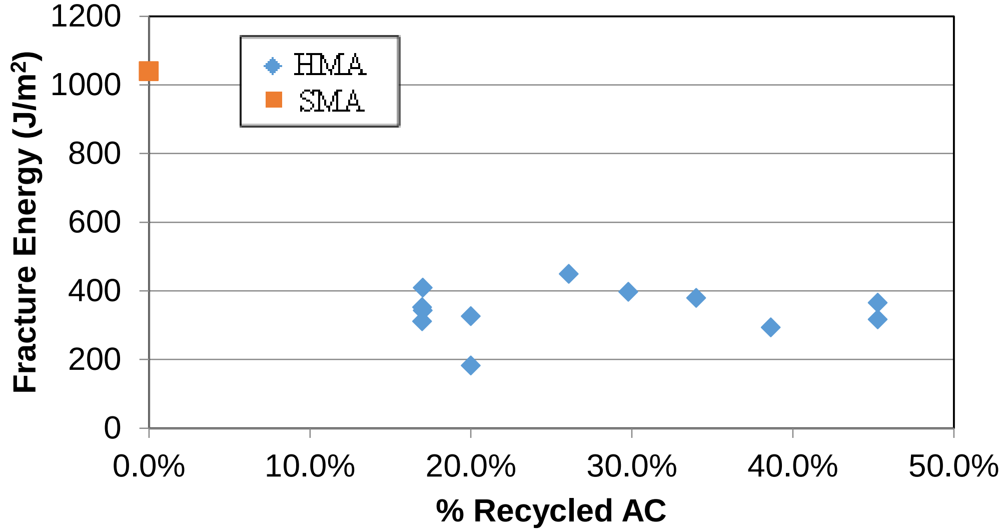

Figure 5.35: Effect of recycled asphalt content (%) on fracture energy

5.4.4 Effect of Voids in Mineral Aggregate (VMA) on Fracture Energy

The comparison between fracture energy and the voids in mineral aggregate (VMA) of each mix can be found in Figure 5.36. All the mixes in this study are three-quarter inch maximum aggregate size; therefore normalizing for recommended VMA values is not beneficial. It should be noted that except for two projects (TH 113 and TH 210) the remaining mixes used in this study were all designed and constructed using the older version of MnDOT 2360 specifications that required a minimum VMA amount. Both of the newer designs that utilized adjusted asphalt film thickness (AFT) specifications had significantly lower VMA amounts.

Figure 5.36 does show a slight upward trend in fracture energy as VMA increases. It should be noted that the results from the Task-3A report showed little to no relationship between VMA and actual transverse cracking amounts for the nine projects studied herein. The addition of different mix sizes to future studies may be advantageous when investigating the effect of VMA.

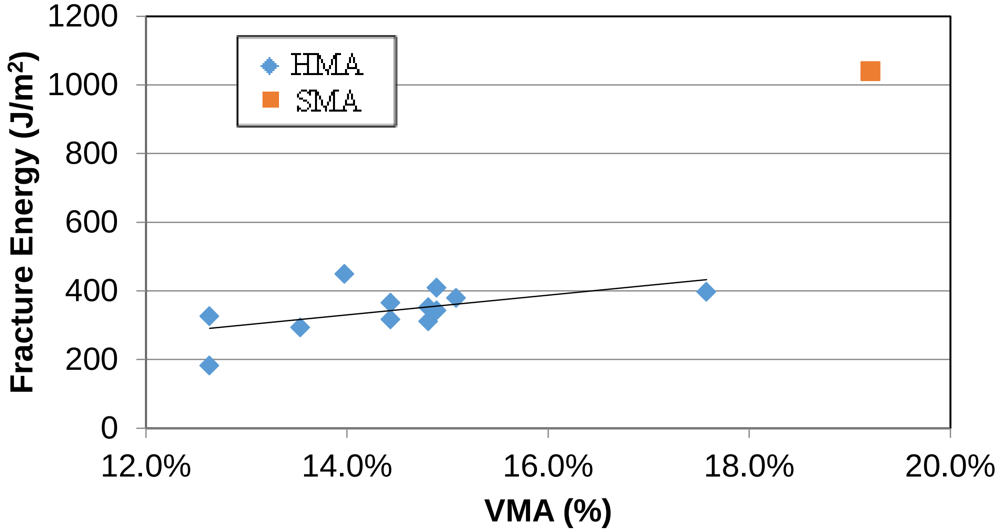

Figure 5.36: Effect of voids in mineral aggregate (VMA) on fracture energy

5.4.5 Effect of Voids Filled with Asphalt (VFA) on Fracture Energy

Figure 5.37 shows the trend that results from the analysis of the voids filled with asphalt (VFA) of each mix. The data in this portion of the study has an identical relationship to that of fracture energy and VMA. While all the mixes in this study do not have the same design traffic level (basis for Superpave VFA recommendations), all of the mixes meet the suggested VFA range for the corresponding traffic level. Further studies will be required to validate if any relationship exists between fracture energy and VFA.

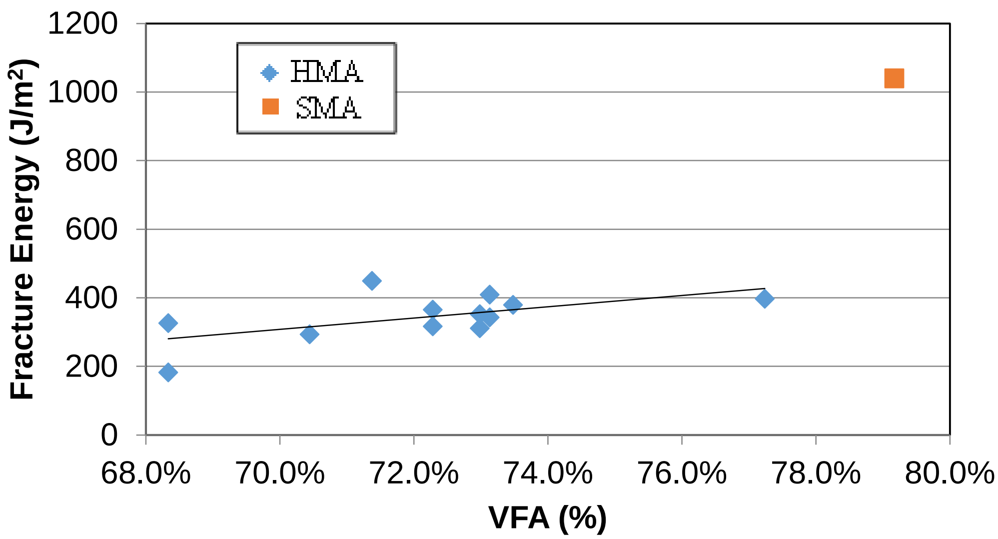

Figure 5.37: Effect of voids filled with asphalt (VFA) on fracture energy

5.4.6 Effect of Adjusted Asphalt Film Thickness (AFT) on Fracture Energy

The adjusted asphalt film thickness (AFT) for various mixes are plotted against the fracture energies from laboratory testing in Figure 5.38. For the mixes designed and produced using the older MnDOT 2360 specifications the adjusted AFT values were calculated using the information from MDRs and the mix test summary sheets (TSS). It should be noted that the TH 212 results were not included in Figure 5.38, as the TSS were not available for this section.

The plot does not appear to indicate an observable trend relating fracture energy and adjusted AFT. As with other parameters the data is still prone to significant scatter and this information should not be used for the purposes of drawing conclusions.

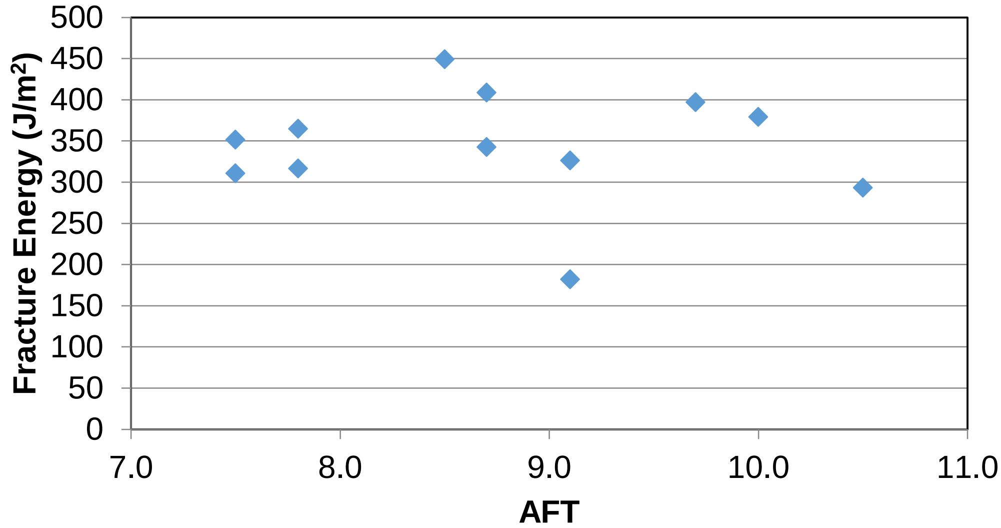

Figure 5.38: Effect of asphalt film thickness (AFT) on fracture energy

5.5 Summary and Conclusions

The Task-3A of this study analyzed the field evaluation results of nine highway projects and sixteen pavement sections. These represent four virgin binder types, three pavement section types and two design traffic levels. The focus of this task was to validate the findings from Task-1 of this project and determine if any correlations exist between mix design parameters and transverse cracking performance in the field. The field evaluation results consisted of crack count surveys by the researchers and historical pavement distress information from the MnDOT Pavement Management System (PMS). The compilation of this data provided researchers with a timeline of transverse cracking performance over the service life of the sections.

The analysis of mix design parameters revealed the following correlations and conclusions:

- PG grade had a slight correlation to improved field performance as the performance grade of the binder progressed in the order of PG 58-28, PG 58-34, PG 64-28 and PG 70-34.

- As PG spread increased, it appeared to correlate well with maximum transverse cracking exhibited by the sections. However, average transverse cracking showed no discernable trend with the PG spread of asphalt binder.
- Asphalt content showed improved transverse cracking resistance as the amount of binder increased in a mix, this was followed by subsequent decrease in performance with further increase in binder amounts.
- Asphalt film thickness correlated to a slight deterioration in field performance as adjusted AFT increased. When plotted versus maximum transverse cracking amount normalized for asphalt layer thickness, this correlation becomes less apparent. This is an indication that this parameter has a negligible impact on transverse cracking performance
- All other mix parameters showed minimal to no correlation with field performance.
- Other than adjusted AFT, normalization for traffic level and asphalt layer thickness had minimal impact on correlations between transverse cracking and mix parameters.

Analysis of construction type versus transverse cracking amounts yielded intriguing results. When observing mix parameters arranged by construction type against cracking performance, two parameters showed a correlation: PG spread and asphalt binder content. The following relationships were observed:

- As PG spread increases, reclaim projects experience significantly better transverse cracking resistance as compared to overlays.
- As asphalt binder content increases, reclaim projects exhibit greater transverse cracking resistance as compared to overlays.

In general, the results indicate that the use of mix design parameters (volumetric controls) as an independent transverse cracking performance predictor may not yield reliable results. These findings are not entirely surprising. The analysis from Task-1 showed that some parameters have potential to be performance predictors but none showed a very strong correlation. The findings from Task-3A reinforces the Task-1 recommendation of using laboratory testing based performance parameter. The findings also reinforce the need for using superior asphalt binder grade in the reclaim sections.

Finally it should be noted that the conclusions regarding the mix parameters in context of different pavement section types (reclaim versus mill and overlay) should be treated preliminary as the number of sections were limited.

The Task-3B of this study analyzed the laboratory testing from field samples of nine highway projects and thirteen pavement sections. These represent four virgin binder types, three pavement section types and two design traffic levels. This task had a primary goal of validating the findings from Task-1, comparing the results of laboratory testing to the findings in Task-3A and determining if any mix design parameters had potential for use as an indicator of fracture energy performance.

It should be noted that the material specifications underwent changes between various mixtures and projects studied herein also the rate of aging of different asphalt binders would be different due to geographical differences and chemical makeup of binders. The impact of these factors is difficult to quantify. These should be considered in any future studies involving field procured specimens.

The analysis of mix design parameters revealed the following correlations and conclusions:

- PG grade had a slight correlation to higher fracture energy as the performance grade of the binder progressed in the order of PG 58-28, PG 64-28, PG 58-34 and PG 70-34. It should be noted that the study did not look at the type of modification for manufacture of -34 binders. A separate analysis is presently underway at UMD to look at effects of binder modification on field cracking performance.
- As PG spread increased, it appeared to correlate with a higher fracture energy. However, only one section was available for the PG spread of 104. This finding should be further validated in future studies.
- PG low temperature grade showed a loose trend of improved fracture energy for PG XX-34 as compared to PG XX-28.
- Asphalt content showed a general increase in fracture energy as the amount of binder increased and reaching a somewhat optimal level before showing drop in performance with further increase.
- Asphalt film thickness did not feature a significant trend. If anything, a lower fracture energy correlated to an increase in adjusted AFT. This is an indication that this parameter has a negligible impact on fracture energy, and thus transverse cracking performance.
- All other mix parameters showed minimal to no correlation with laboratory testing.

In general, the results indicate that the use of mix design parameters as an independent fracture energy performance predictor is not recommended. These findings are not entirely surprising. The analysis from Task-1 showed that some parameters have potential to be performance predictors but none showed a very strong correlation. The findings from Task-3B reinforce the Task-1 recommendation of using laboratory testing based performance parameter. While some parameters indicate slight trends with increased fracture energy, none are definitive. PG grade, PG spread and asphalt content all show encouraging trends. These will be observed closely in future studies.

CHAPTER 6: IMPLEMENTATION ASSESSMENT AND DRAFT PERFORMANCE BASED SPECIFICATIONS (TASK-4)

6.1 Introduction

The purpose of this task in the research study was development of the draft performance testing based specifications to supplement the currently used asphalt mixture specifications. Furthermore, the task also undertook effort of determining the necessary information for implementation of the test in terms of equipment, sampling needs, specimen preparation and personnel needs. During the course of the project, a number of activities were undertaken to realize this objective. Significant effort was in terms of communications between the university researchers and MnDOT staff from Office of Materials and Road Research (OM&RR). The other task objectives were realized in conjunction with the laboratory testing during the course of this project as well as through additional testing that was conducted on request of OM&RR.

Through the research efforts of this projects as well as other studies, the fracture energy of asphalt mixture as measured using the disk-shaped compact tension (DCT) test has continued to show very promising results to serve as a performance parameter. This became evident during early phase of the study, thus over the course of over last 2 years various activities to implement DCT specifications have occurred. The summary of various activities and corresponding outcomes associated with this task are described next.

6.2 DCT Fracture Energy Specifications

The DCT fracture energy based transverse cracking performance specifications were proposed from the efforts of the two phases of the Low Temperature Cracking Pooled Fund Studies (Marasteanu et al., 2008 and 2012). A synthesis of the lab performance test that preceded this study recommended use of those specifications. The testing efforts from this study as well as other studies (by researchers as well as others) have continued to increase the confidence in use of DCT fracture energy as performance indication measure. For example the testing of field cores from TH 371. These are briefly discussed in following subsection.

6.2.1 TH371 Testing

A set of samples were received from MnDOT OM&RR for project on trunk highway TH371 near Brainerd, MN. Cored field samples were received from three different sections on TH371, which were constructed in 2004 and 2005. Since then, excessive cracking has been found at specific locations of the highway near reference posts (RP) 17-21. Wearing and non-wear course samples were received from RP 6, RP 17, and RP 21.5. Cracking was found most prevalent near RP 17 and RP 21.5.

Figure 6.1 and Figure 6.2 provide fracture energy and peak load results for TH371 samples. Fracture energies decreased further towards the reference post 21.5. As previously mentioned, excessive transverse cracking was found between RP 17 and RP 21.5. These fracture energies are below the 400 J/m^2 threshold for wear course, indicating poor performance. RP 6 sample fracture energies exceeded the 400 J/m^2 threshold for both wear and non-wear courses. Peak load results showed very small differences between all samples. The peak load form the DCT fracture energy test usually shows good correlation to the tensile strength of the mixture, the lack of distinction in peak load between various samples indicate that strength may not be an adequate transverse cracking performance parameter. This reaffirms the findings from task-1 of this project. It should also be noted that RP 17 and 21.5 samples had slightly higher peak loads. The higher peak loads and lower fracture energies indicate the mixture used near RP 17 and 21.5 is a stiff, brittle mix compared to RP 6. The stiffer and more brittle mixtures is anticipated to have led to early and excessive cracking.

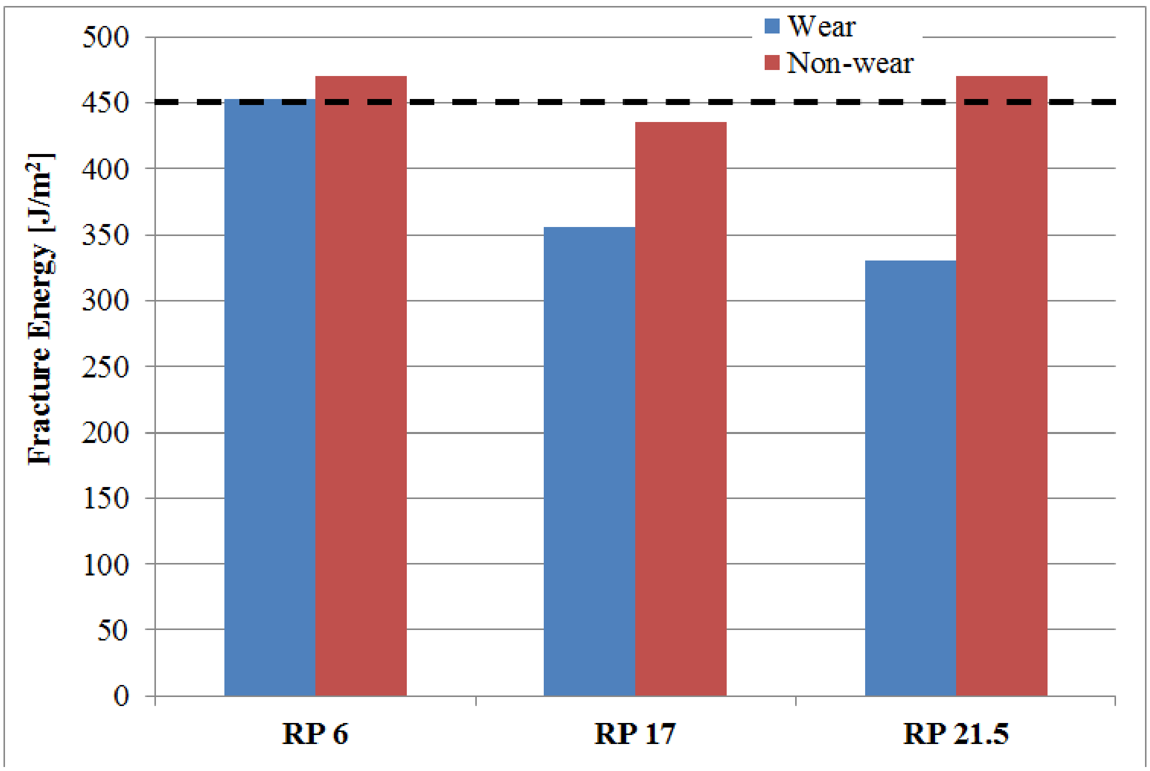

Figure 6.1: DCT fracture energy results for TH371 samples

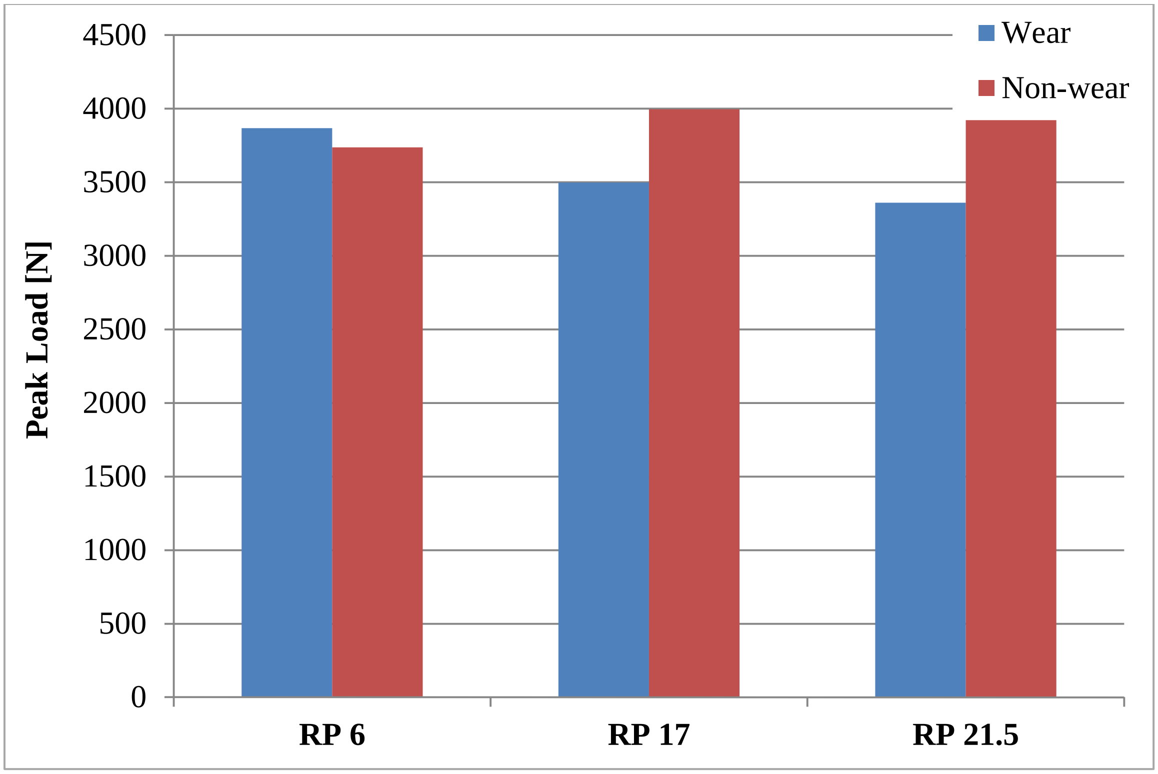

Figure 6.2: DCT peak load results for TH371 samples

The number of cracks found on TH371 near the reference posts can be seen in Table 6.1 along with their corresponding fracture energies. It should be noted that the fracture energy values decrease significantly from reference post 6 to reference post 21.5. This also corresponds to an increasing amount of cracks seen in the field, especially in the south bound lane. This further indicates the DCT test has the ability to correlate laboratory data to actual field cracking performance.

Table 6.1: DCT fracture energy results versus cracking seen in the field on TH371

RP	North Bound Crack Count	South Bound Crack Count	Fracture Energy [J/m^2]	
			Wear Coarse	Non-wear
6	3	4	453.44	470.02
17	12	8	356.18	435.05
21.5	10	57	330.59	470.45

The results like the ones from TH371 field cores show that the threshold value of 400 J/m^2 for the DCT fracture energy continues to be a reliable indicator of the transverse cracking performance of the asphalt mixture. A majority of the fracture energy results on the field procured specimens tested in this study were very close to the threshold value of 400 J/m^2. With exception of two sections all others showed average fracture energy to be within 20% of the 400

J/m^2 making it difficult to see large enough variation between good and poor performing sections. While this is the case, for projects with multiple observation sections (such as TH1 or TH113) it was evident that mixes with higher fracture energies showed better transverse cracking performance. The subsequent section describes the field performance and laboratory testing results for the two sections of TH113.

6.2.2 TH113 Field Performance and DCT Fracture Energy Results

Please note that an in-depth evaluation of the field performance and laboratory testing for field samples was undertaken in Tasks 2 and 3 of this project. The details on the sampling, testing and analysis of the data are presented in the sections relevant to tasks 2 and 3. This subsection provides brief summary of those efforts.

As part of this study two sections were identified on TH113, one corresponding to 1000 ft. of pavement located near RP5 and other one near RP10. The transverse cracking performance of the two sections over eight years of service are presented in Figure 6.3. It can be clearly seen from this plot that pavement near RP 5 significantly outperformed pavement neat RP 10 in terms of transverse cracking performance. The fracture energy results of the cored samples from these sections are presented in Figure 6.4. As observed clearly in this plot the DCT fracture energy of the wear course mixture from good performing section (RP5) is significantly higher than that from the poor performing section (RP10).

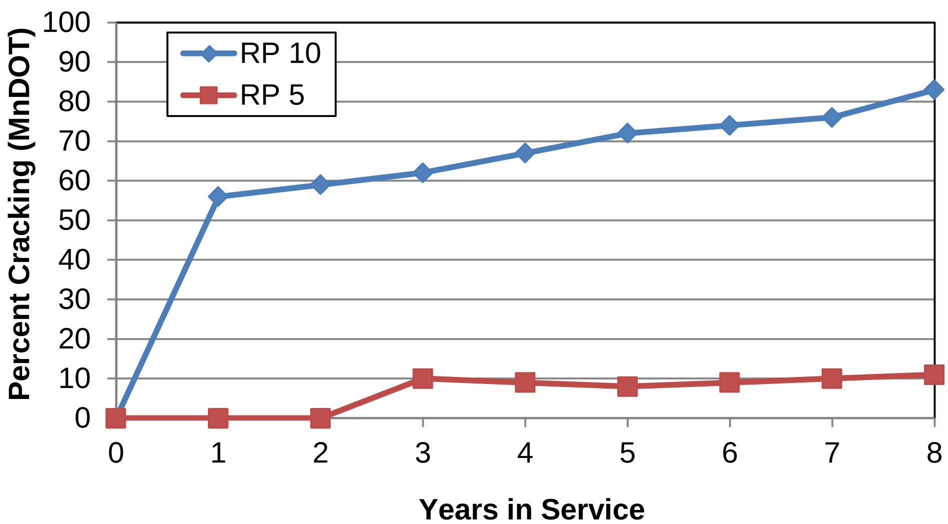

Figure 6.3: Transverse Cracking Performance of TH113 Pavement Sections

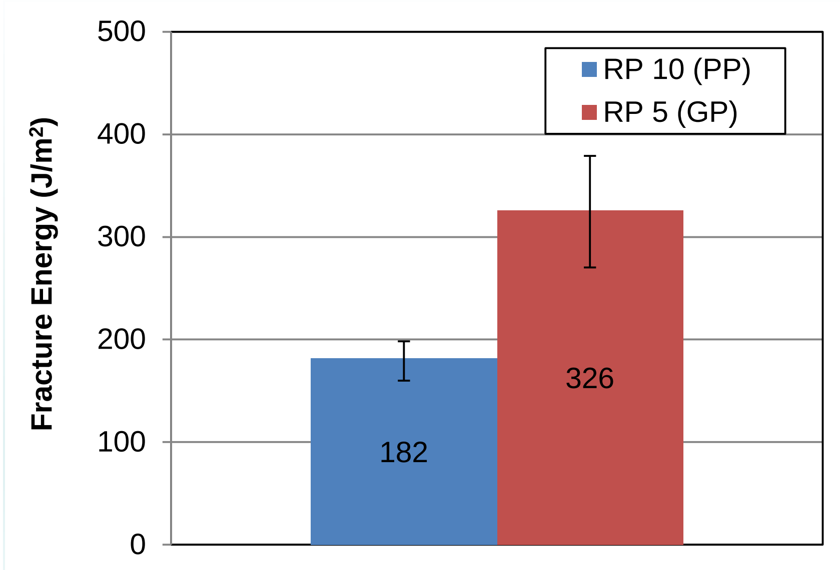

Figure 6.4: DCT Fracture Energies of TH113 Pavement Sections

6.2.3 Recommended Specifications

On basis of the testing results and the findings of previous studies, as well as laboratory testing and data analysis from this study, the DCT fracture energy based low temperature cracking performance specifications presented by Marasteanu et al. (2012) are recommended to be implemented. The specification consists of a tiered approach, whereby depending on the design traffic levels, different fracture energy thresholds are recommended. Table 6.2 shows the proposed thresholds for transverse cracking performance specifications.

Table 6.2: Recommended DCT Fracture Energy Thresholds for Transverse Cracking Performance Specification

	Project Criticality / Traffic Level		
	High (> 30M ESALs)	Medium (10 – 30M ESALs)	Low (< 10M ESALs)
Min. Required DCT Fracture Energy (J/m^2)	690	460	400

Please note that the original specifications developed from two phases of the Low Temperature Cracking Pooled Fund Study (Marasteanu et al., 2012) also recommended that the IlliTC thermal cracking prediction system be employed for asphalt mixtures to be used on high traffic roadways. At present, it is recommended that initial implementation be conducted with only

DCT fracture energy and subsequently the IlliTC requirements be introduced. On the basis of this project, IlliTC should be continue to be investigated to see if it is a viable for future implementation in conjunction with the DCT fracture energy specification. IlliTC has the capability to simulate thermal cracking performance with specified asphalt mixture parameters, over a specific performance period, with consideration of the cumulative damage effects. Furthermore, it should be noted that the initial implementation of the performance specifications be limited to wear course mixes that are used in new construction, reclaim or thick mill and overlay (> 3.5 inch) construction. That is, at present the specification is not recommended to be applied to mixes used in thin overlay applications.

6.3 Implementation of Performance Specifications

The four major areas of exploration to full-scale implementation of laboratory performance test as part of asphalt mixture acceptance process are: (a) testing equipment; (b) test procedure; (c) sampling needs and specimen preparation; and, (d) personnel requirements. Preliminary assessment in these areas was conducted over the course of this project. Please note that a parallel project has been underway since last nine months that has been focused on much more detailed assessment of various implementation tasks as well as development of refined testing protocol, sample collection, specimen preparation and conditioning, and data analysis. A brief description for various aspects regarding the implementation of DCT fracture energy performance test are presented here, however the reader is encouraged to refer to the task reports and findings from the MnDOT Work Order 162 for more current and in-depth information.

6.3.1 Specifications for Specimen Requirements and Testing Procedures

As described earlier the ASTM D7313 forms the basis for conducting the DCT fracture energy tests. However, in order to streamline the specimen preparation and temperature conditioning procedures as well as to make the testing procedure better suited for routine usage the MnDOT OM&RR staff has worked extensively over the course of last two years to develop an enhanced version of specifications (MnDOT modified version of ASTM D7313). Please note that the researchers were involved in this process in role of consultants and participated via discussions and meetings. Also note that these specifications are essentially for the purposes of specimen preparation and testing protocols the specifications for material performance requirements are separate and as discussed in previous section. The current version of this specification is attached as an appendix to this report.

6.3.2 Testing Equipment

The general requirements for the necessary testing equipment in order to conduct disk-shaped compact tension fracture tests are laid out in the ASTM D7313 test specifications. The test procedure recommends use of a loading device with capabilities to have close-loop control in order to maintain a constant rate of crack mouth opening. The equipment also needs to have capability for the conditioning of the test specimens at a constant temperature and maintaining constant specimen temperature over a duration of time. While any axial loading system with close-loop control capabilities and a temperature chamber meets these requirements, it should be noted that at present one stand-alone device for DCT testing is readily available. At the time of writing of this report the MnDOT OM&RR and two consulting labs (American Engineering Testing and Braun Intertec) have the stand-alone DCT testing device in Minnesota, and nation-wode over 10 such devices are avialble at various laboratories. Furthermore, within Minnesota and in neighboring states a number of other organizations have DCT testing capabilities (University of Minnesota Duluth, MTE Services Inc., S.T.A.T.E. testing laboratories, Iowa Department of Transportation, Payne and Dolan etc.).

6.3.3 Sampling Needs and Specimen Preparation

The implementation of the DCT fracture energy specifications are recommended to be conducted in a manner similar to those undertaken with implementation of the QA process based asphalt mixture volumetric specifications. That is, an approval process for acceptance of the mixture is required prior to actual production and during the course of production proof-testing is conducted on mixture to ensure that the mixture meets the specifications. As indicated before, at the time of writing of this repot another research study is ongoing that is focused on the implementation of the DCT fracture energy specifications. That study is in process of formalizing the sampling needs for the specification. The preliminary recommendations from the current study in terms of sampling needs are described next.

As part of the mix design acceptance process, four samples should be submitted by the contractor to the MnDOT. The samples are to be laboratory mixed and compacted using the gyratory compactor following the procedures same as those used for the moisture sensitivity determination (commonly referred to as TSR specimens), in other words by following the procedures described in the AASHTO T283 specifications. During the course of mix production, samples should be collected for DCT testing. The number of samples and their timing is currently being evaluated through the implementation study. The preliminary recommendation is that at least one set of sample (sufficient to manufacture four DCT specimens) be collected for each day of paving.

The general specimen requirements for the DCT testing is entailed in ASTM D7313 test specifications. During the course of present study the researchers worked extensively with the MnDOT OM&RR staff to help in procuring the necessary equipment and tools for preparation of the DCT specimens. A brief specimen preparation guide for DCT specimen preparation is attached as an appendix to this document.

6.3.4 Personnel Requirements

The major activities associated with the DCT fracture energy testing can be broken down into three phases: (1) Specimen preparation; (2) DCT testing; (3) Data analysis and reporting. The first phase is the most labor intensive as it involves the process of compacting the asphalt mixture and a series of cutting and coring steps to get the specimen ready. Once the specimen is prepared into the desired geometry, this step also involves gluing of metal knife-edges for attaching the displacement transducer as well as measurement of the specimen dimensions. On basis of the time durations experienced by the researchers it is not uncommon to take approximately eight to twelve hours for preparation of specimens. Please note that there is significant down-time associated with this process (for example, heating of mix, drying of specimens etc.). Thus, in terms of working hours it is estimated that a trained personnel can prepare one DCT specimen in approximately 30 minutes. Thus, total working time for each type of mix is approximately 2 hour. This time can be further reduced through improved scheduling whereby multiple mixtures are being converted into test specimens and more than one personnel is involved in the process so that several activities can be conducted in parallel to each other.

The time estimate for the actual testing process depends on the time spent in temperature conditioning of the specimens. Following the current MnDOT modified version of the procedure, the estimated time spent by a trained personnel is approximately 2 hours per mixture (assuming that multiple mix types are being tested simultaneously). Finally, to document the results and compute images of broken specimens a trained personnel would require approximately 1 hour per mix type. In summary the current preliminary estimate for the required personnel effort for conducting DCT fracture energy tests on plant produced loose mixture is estimated to be 5 hours per mixture (assuming four replicate specimens). Please note that this estimates assume that more than one mix type is being testing simultaneously. Typically 3 mix types is found to have been an optimum number for researchers.

6.4 Summary

The Task-4 of the present study was designed with objective of recommending the laboratory performance testing based specifications for asphalt mixtures to improve the field transverse cracking performance. Through efforts of previous tasks of the current study as well as other parallel and previous studies, the fracture energy of asphalt mixtures was determined to be a viable transverse cracking performance parameter. The fracture energy measurement is recommended to be conducted suing the using the disk-shaped compact tension (DCT) test. It is recommended that the fracture energy thresholds proposed at the conclusion of the low-temperature cracking pooled fund study be adopted and be used as additional requirement to the current MnDOT 2360 specifications. It is also recommended that the initial implementation of this proposed specification be done on wear course mixes that are planned to be used for new construction, full reconstruction, reclaim and thick overlay projects. Several modifications to the current ASTM D7313 methods have been developed through efforts of MnDOT OM&RR, these be adopted during the continued implementation efforts. Finally, there is an ongoing research project (MnDOT Work Order 162) that is focused on the DCT implementation efforts, readers are recommended to refer to the memos and reports form that study for further information.

CHAPTER 7: SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

A brief summary of the research conducted through this project can be presented by describing key highlights of the efforts from each of the project tasks. The results of Task-1 concluded that the indirect tensile strength (ITS) of the asphalt mixes, as determined using the AASHTO T-283 specifications, is found to be a poor measure of pavement cracking performance. Tasks 2A and 3A made comparisons between field cracking performance and asphalt mixture attributes as well disk-shaped compact tension (DCT) fracture energy measurements. As a part of Tasks 2B and 3B, the performance testing and comparison to field data affirmed the potential for using the DCT test as a performance indicator through the apparent relationship between decreasing fracture energy and increasing transverse cracking amounts. Finally, Task-4 utilized the conclusions of previous tasks to determine the fracture energy of asphalt mixtures to be a viable transverse cracking performance parameter. The project proposes use of performance based specifications by recommending fracture energy from the disk-shaped compact tension (DCT) to be added to currently practiced asphalt mixture specifications. Furthermore, the task 4 also undertook effort of determining the preliminary information for implementation of the test in terms of equipment, sampling needs, specimen preparation and personnel needs.

Individual task summaries and conclusions are provided in the subsequent sub-sections. Afterward, recommendations for future research are presented.

7.1.1 Task-1

The Task-1 (Analysis of Laboratory Test and Field Performance Data) of the MnDOT research contract 99008 undertook three primary research efforts:

(1) Development of a comprehensive database that includes asphalt material property data (mix design records), bituminous pavement construction information (SP information, location, construction year) and the pavement management information (section locations, survey years, cracking data);

(2) Determination of whether the indirect tensile strength (ITS) from the modified Lottman test (AASHTO T-283) can be used as a cracking performance measure; and

(3) The effects of mix design parameters (mix volumetrics, mix design (traffic) level, asphalt binder amounts and grades, use of recycled materials) on the pavement cracking performance.

Due to the large quantity data analyzed, it was necessary to use statistical analysis for determining effects of one parameter on other. Based on the three efforts listed above, a number of findings were made. The findings from this effort allow identification of mix design parameters that affect pavement cracking performance. The study also determined the effects of the mix design choices on the cracking performance such as, use of -28 grade asphalt binder as compared to -34 grade binder.

The main conclusion that directed the research for the other tasks in this project was the indirect tensile strength (ITS) of the asphalt mixes, as determined using the AASHTO T-283 specifications, was found to be a poor measure of pavement cracking performance. Additionally, the key conclusions from analysis of mix design parameters drawn from this study are as follows:

- A higher percentage of crack free pavements were represented by asphalt mixes that have lower adjusted asphalt film thickness (AFT) and higher voids in mineral aggregates (VMA). For pavements that have cracks present in them, neither adjusted AFT nor VMA showed consistent trends.
- Asphalt binder grade has a significant effect on the pavement cracking performance. Mixes containing -34 asphalt binders have significantly greater amount of crack-free pavements as compared to mixes containing -28 binders. Fewer percent of pavements with significant amounts of transverse cracking are represented by mixes with -34 binder grades as compared to those with -28 binder grades.
- The amount of asphalt binder has a significant effect on field cracking performance. The mixes with higher asphalt content showed lower amounts of cracking.

7.1.2 Task-2A

The Task-2A of this study focused on the field evaluation of nine highway sections. The pavement study sections were evaluated to conduct crack counts as well as visual distress survey. The data collected during the site visits is summarized in section 3.2.4 of this report, the raw crack count data is provided as Appendix-C to this report. Furthermore, the locations for obtaining cored samples for performance testing were also identified. Using the sample collection information and on basis of the construction drawings, field sampling plans were developed and delivered to MnDOT staff. These plans are also attached to this report as Appendix-D.

A number of cracking performance measures were developed through Task-1 of this project (Appendix-G). Those performance measures were utilized in conjunction with the pavement management data and information from field visits to quantify the cracking performance of pavement sections. The information collected and processed through this task was utilized in Task-3A and 3B to make comparisons between field cracking performance and asphalt mix attributes as well as disk-shaped compact tension (DCT) fracture energy measurements.

The results of this task compare the classification of pavement performance through the use of various cracking performance measures without sole reliability on one measure. While detailed analysis of the data was conducted through Task-3A, some general observations from the cracking performance measures and sites visits are as follows:

- The average of the maximum cracking amount (MTCTotal) of all 18 study sections is approximately 7% per year of service. This information can be used to determine the number of years of service at which the pavement is expected to reach the state of 100% transverse cracking. On an averaged basis, using data from 18 pavement sections studied herein, approximately 14 years of service to reach 100% transverse cracking is obtained. The shortest life as seen from the study sections is expected to be 6 years.

- For the sections studied in this project the maximum cracking rate (MTCRTotal) is observed to be as high as 82% per year with average of 30.6% per year.
- The average of the average transverse cracking amounts (ATCTotal) for all 18 sections is approximately 30.7%. This measure indicates the average amount of cracking that would present on any section during the course of its service life.
- The asphalt layers on reclaimed sections show lower amount of cracking and delayed cracking as compared to mill and overlay sections on the same stretches of highways. It should be noted though that the reclaim sections consists of greater asphalt layer thicknesses (3 – 4 inch) as compared to mill and overlay sections (1-1/5 – 2-1/2 inch).
- The pavement sections consisting of asphalt overlay on PCC pavements showed significant reflective cracking within first year of service. Once all joint/cracks reflected into the overlay minimal additional cracking is observed.

7.1.3 Task-2B

The Task-2B of this study focused on the performance testing and comparison to field data for nine highways. During this task the field cores from each highway section, 13 sections in total, were tested using the disk-shaped compact tension test. The results of this effort are summarized throughout the various sections of this report. Data was compared to field performance using various transverse cracking measures, in an effort to reduce any reliability on a potentially misleading measure. These measures were developed through Task-1 of this project and modified as required during the analysis process. Those performance measures were utilized in conjunction with the pavement management data and information from field visits to quantify the cracking performance of pavement sections. This data was presented in several fashions, considering both traffic level of the section and pavement construction type.

The main conclusion of this task was the apparent relationship between decreasing fracture energy and increasing transverse cracking amounts is apparent for various measures of cracking performance. This reaffirms the potential for using the DCT test as a performance indicator.

7.1.4 Task-3A and 3B

The Task-3A of this study analyzed the field evaluation results of nine highway projects and sixteen pavement sections. The Task-3B of this study analyzed the laboratory testing from field samples of nine highway projects and thirteen pavement sections. These represent four virgin binder types, three pavement section types and two design traffic levels. The focus of Task-3A was to validate the findings from Task-1 of this project and determine if any correlations exist between mix design parameters and transverse cracking performance in the field. Similarly, the primary goal of Task-3B was validating the findings from Task-1, comparing the results of laboratory testing to the findings in Task-3A and determining if any mix design parameters had potential for use as an indicator of fracture energy performance.

The analysis of mix design parameters in both tasks 3A and 3B revealed the following correlations and conclusions that validate the findings from Task-1:

- PG grade had a slight correlation to improved field performance as the performance grade of the binder progressed in the order of PG 58-28, PG 58-34, PG 64-28 and PG 70-34. It should be noted that the study did not look at the type of modification for manufacture of -34 binders. A separate analysis is presently underway at UMD to look at effects of binder modification on field cracking performance.
- As PG spread increased, it appeared to correlate well with maximum transverse cracking exhibited by the sections. However, average transverse cracking did not show a consistent trend with PG binder spread.
- Asphalt content showed improved transverse cracking resistance as the amount of binder increased in a mix, this was followed by subsequent decrease in performance with further increase in binder amounts.
- Asphalt film thickness correlated to a slight deterioration in field performance as adjusted AFT increased. When plotted versus maximum transverse cracking amount normalized for asphalt layer thickness, this correlation becomes less apparent. This is an indication that this parameter has a negligible impact on transverse cracking performance.
- All other mix parameters showed minimal to no correlation with field performance nor laboratory testing.
- Other than adjusted AFT, normalization for traffic level and asphalt layer thickness had minimal impact on correlations between transverse cracking and mix parameters.

Analysis of construction type versus transverse cracking amounts yielded intriguing results. When observing mix parameters arranged by construction type against cracking performance, two parameters showed a correlation: PG spread and asphalt binder content. It should be noted that the conclusions regarding the mix parameters in context of different pavement section types (reclaim versus mill and overlay) should be treated preliminary as the number of sections were limited. The following relationships were observed:

- As PG spread increases, reclaim projects experience significantly better transverse cracking resistance as compared to overlays.
- As asphalt binder content increases, reclaim projects exhibit greater transverse cracking resistance as compared to overlays.

There were similar findings for increased asphalt binder content and increased PG spread amongst tasks 1, 3A and 3B relating to better pavement performance. Similarly, superior pavement performance was seen in sections with -34 binders as compared to -28 binders. Contrarily, some mix design parameters saw discrepancies in results between tasks, such as AFT and adjusted AFT.

In general, the results indicate that the use of mix design parameters (volumetric controls) as an independent transverse cracking nor fracture energy performance predictor may not yield reliable results. These findings are not entirely surprising. The analysis from Task-1 showed that some parameters have potential to be performance predictors but none showed a very strong correlation. The findings from Task-3A reinforces the Task-1 recommendation of using laboratory testing based performance parameter. The findings also reinforce the need for using superior asphalt binder grade in the reclaim sections. From Task-3B while some parameters indicate slight trends with increased fracture energy, none are definitive. PG grade, PG spread and asphalt content all show encouraging trends. These will be observed closely in future studies.

7.1.5 Task-4

The Task-4 of the present study was designed with objective of recommending the laboratory performance testing based specifications for asphalt mixtures to improve the field transverse cracking performance. Through efforts of previous tasks of the current study as well as other parallel and previous studies, the fracture energy of asphalt mixtures was determined to be a viable transverse cracking performance parameter. The fracture energy measurement is recommended to be conducted suing the using the disk-shaped compact tension (DCT) test. It is recommended that the fracture energy thresholds proposed at the conclusion of the low-temperature cracking pooled fund study be adopted and be used as additional requirement to the current MnDOT 2360 specifications. It is also recommended that the initial implementation of this proposed specification be done on wear course mixes that are planned to be used for new construction, full reconstruction, reclaim and thick overlay projects. Several modifications to the current ASTM D7313 methods have been developed through efforts of MnDOT OM&RR, these be adopted during the continued implementation efforts. Finally, there is an ongoing research project (MnDOT Work Order 162) that is focused on the DCT implementation efforts, readers are recommended to refer to the memos and reports form that study for further information.

7.2 Recommendations

The key recommendations from the research efforts of this study are as follows:

- The development of a comprehensive database required developing an extensive search algorithm to map the cracking data from pavement management highway sections onto the material records from the laboratory information system and the construction records. If the future versions of the pavement management system can include a variable that tracks the highway construction information (for example, project SP), the development of a comprehensive database, such as one developed in this study, will require significantly fewer amount of human resources and computational efforts.
- The asphalt binder amount and grade play an important role in the cracking performance of bituminous pavements and overlays. The asphalt binder grade recommendations along with the potential for use of a minimum asphalt binder amount in the specifications should be reevaluated.
- It was not possible to analyze the effects of the amount of recycled materials on pavement cracking performance. Future research projects should evaluate this effect.
- The impacts of binder age and core location have an unknown influence on the fracture energy discussed in this report. A core from the beginning of the project and a core from the end of the project can have significant differences. Similarly, a binder from a project in 2010 will have aged differently than a project in 2003. The impacts of both of these factors are not well documented at this point. Future research should recognize these components when using field procured samples.
- TH 212 performed at an exceptional level during testing, exhibiting an average fracture energy of 1,040 J/m^2. This is far greater than any other section in this study and well above the 400 J/m^2 threshold. Being that this section has experienced zero transverse cracking over the six year service life, it would appear to further validate the use of this threshold.

    - It should be noted that this section was constructed with the only stone-matrix asphalt (SMA) mixture in this study as well as the only new construction project.
- The relationship between PG grade did not look at the type of modification for manufacture of -34 binders and should be evaluated in future research.